THE UNIVERSITY OF ALBERTA

# APPLICATIONS OF THE TENSILE SPLITTING TEST TO ASPHALT CONCRETE MIXTURES AT LOW TEMPERATURES

by

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled

APPLICATIONS OF THE TENSILE SPLITTING TEST TO ASPHALT CONCRETE MIXTURES AT LOW TEMPERATURES

in partial fulfilment of the requirements for the degree of Master of Science.

## ABSTRACT

An evaluation of the tensile splitting test to measure the tensile strength and strain at failure of Marshall cylindrical specimens at low temperatures is presented. Asphalt from two sources, different asphalt contents and temperatures from - 10° F to 20° F provided the variables in the testing program. All specimens were tested at a constant rate of deformation.

The results of the tensile splitting tests show that the strength of the asphalt mixtures can be used to differentiate between asphalt sources on the basis of stress, strain and stiffness characteristics at failure.

Asphalts used in pavements with the higher cracking frequencies show less strain, greater strength and greater stiffness at failure than asphalts associated with pavements showing lower cracking frequencies.

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CHAPTER 1

# INTRODUCTION

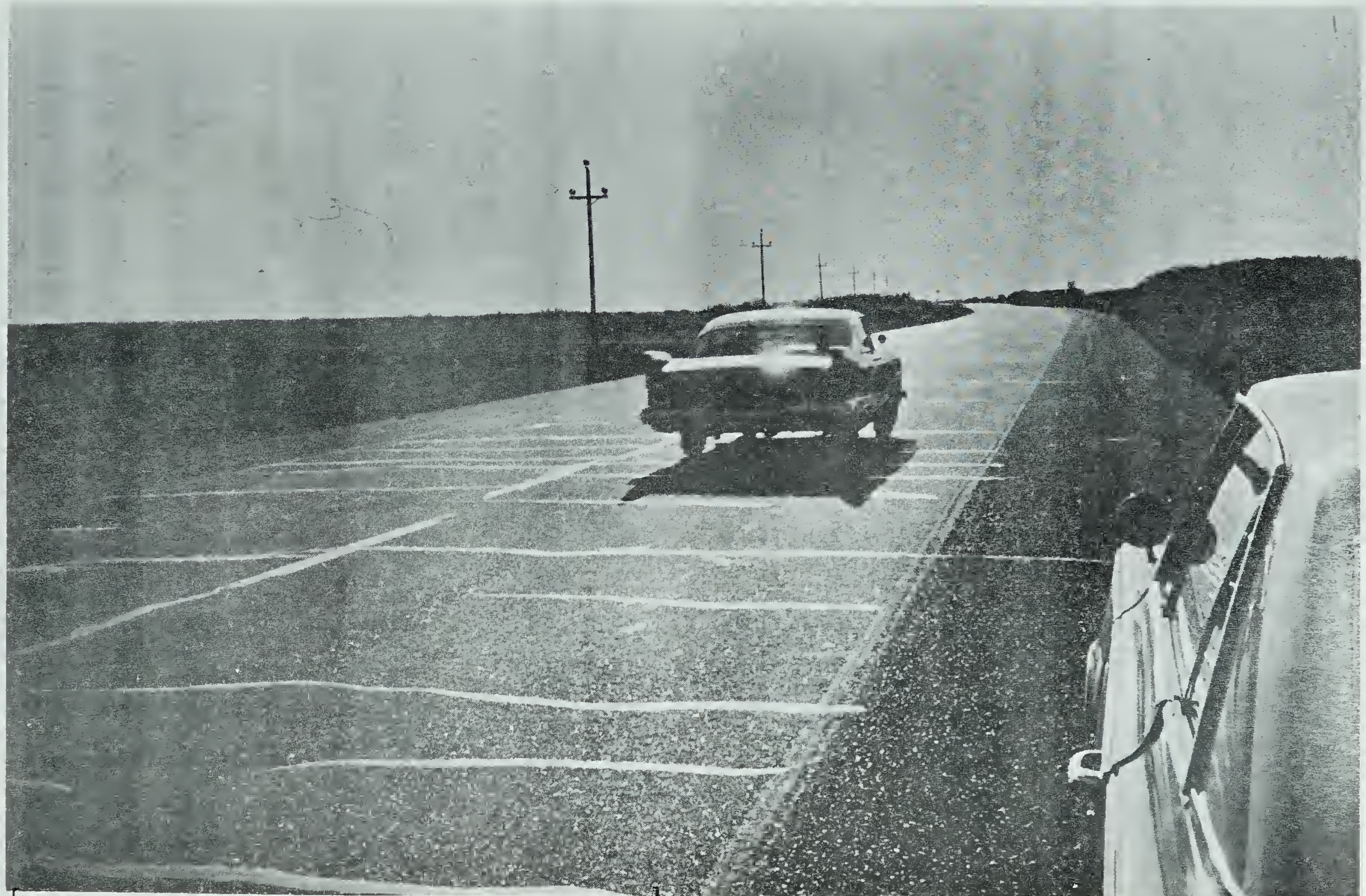
## Background

The highways of our modern transportation systems require smooth riding surfaces for the safe passage of high volumes of heavy traffic. Modern methods of highway design have utilized asphalt concrete pavements as a means of providing these smooth riding surfaces on many of the nation's highways. Over the past years, the appearance of a regular network of transverse cracks has been observed, which extend through the asphalt concrete surface, the subbase, and even into the subgrade itself. Since the presence of transverse cracks is not, initially, the result of progressive failure, which can be attributed to loss of support of the underlying materials there is usually no decrease in pavement performance. However, in those areas, where highways are constructed on soils susceptible to swelling, the existence of transverse cracks will allow moisture to penetrate to the subgrade and eventually cause swelling or warping at the cracks, with a resulting loss in pavement performance.

In the Province of Alberta, many of the highways are surfaced with asphalt concrete pavements, and because

of the scarcity of other materials, are of necessity, constructed on soils susceptible to swelling. Over the past few years, the existence of transverse cracking on many of the recently constructed provincial highways, has been associated with losses in pavement performance and the reasons for cracking are being investigated by the Alberta Cooperative Highway Research Program. Figure 1 shows a photograph of the typical, high frequency regular transverse cracking that occurs on many of the highways in the Province of Alberta.

Up to this time, research under the ACHRP has been concentrated in selected study areas which contain cracked and uncracked sections. Examination of the Marshall Mix Designs used in the selected areas do not show any specific mechanical property that may be related to the cracked or uncracked sections. Field investigations have included mapping of cracked areas, coring of the areas for samples of the surface, subbase and subgrades. Laboratory investigations have included tests on the properties of recovered asphalt samples, closed system freezing of undisturbed subgrade samples to -15°F for one cycle with volume change measurements, and tests on sub-base materials. The results of these testing programs are not conclusive. Tests on the asphalt indicate that high cracking frequencies are usually associated with particular asphalt suppliers, although exceptions have



[PHOTO SUPPLIED BY ALBERTA RESEARCH COUNCIL]

FIG. 1 HIGH FREQUENCY, REGULAR TRANSVERSE CRACKING

been noted. For the remaining tests on the subbase and subgrade materials, the results could not be related to cracking frequency.

In the Province of Alberta, transverse cracking patterns usually appear during the winter months, and are therefore attributed to thermal effects. Shields (1964) has postulated that:

> "the main causative factor is the thermal regime adjacent and within the pavement promoting differential movements that result in stress concentrations within the bituminous surface."

The mechanisms for the formation of cracks in asphaltic pavement are described in detail by Shields and may be classed as follows:

(a) Cracks in the pavement surface only are caused by thermal stresses which exceed the tensile strength of asphalt concrete surface.

(b) Cracks which extend through the subbase and subgrade are caused by the shrinkage of the subgrade which induce stresses in the pavement greater than the tensile strength of the asphalt concrete surface.

Other investigators have studied the reasons for cracking and Domaschuk (1964) suggests that cracking is caused by volume change tendencies in the asphaltic concrete itself. Monismith (1965) has shown by laboratory tests that temperature changes, particularly in the range below 0°F induce stresses that considerably exceed the breaking strength of the asphaltic concrete.

Thus the exact mechanism of crack formation is not known and the inconclusive results of the extensive research program previously described by Shields (1964) indicate that additional research is warranted before the reasons for cracking are fully understood.

## The Scope of the Thesis

This thesis is part of the continuing research program and was concerned with the stress strain characteristics of asphaltic concrete cylindrical specimens at low temperatures. The thesis was divided into three stages:

### Stage 1

Further investigations were carried out on the mix designs of the six study areas as explained in Shields (1964) report. It was proposed to study the mix designs with respect to the surface area of the aggregate, film thickness, bitumen index, and a voids-bitumen index ratio as proposed by Goode (1965). This

further study presented a different approach to that of analyzing the Marshall Method of Mix Design by the conventional criteria of per cent air voids, per cent voids in mineral aggregate, stability, and flow values as suggested by the Asphalt Institute (1962).

Stage 2

This stage of the thesis evaluated a test method which would provide a measure of the tensile stress and strain across the diameter of asphalt specimens. The tensile splitting test was used to induce a tensile stress across the specimen and the corresponding strains were measured with a Tuckerman Optical Strain Gauge.

Stage 3

The testing program was designed to show any differences in the stress strain characteristics of specimens formed from two asphalt sources. The two asphalts sources were selected because each has been used in pavements which have shown very different transverse cracking frequencies during their service life.

## Limitations to the Thesis

This brief study of the stress strain characteristics of asphalt materials using the tensile splitting test has several limitations. Since the tensile splitting

test is a destructive test, the extremely large number of specimens that would be required for a comprehensive testing program is beyond the scope of this study. The results obtained are restricted to a single mix design and will show the differences between two asphalts over a range of asphalt contents. The other limitations pertain to the viscoelastic nature of asphalt materials. The stress strain characteristics of asphalt materials are dependent upon the rate of loading and temperature and the magnitude of these variables are, in turn, restricted by the range of deformation the Tuckerman Strain Gauge is capable of measuring. Thus, the scope of this investigation is restricted by the equipment used to a single rate of deformation and a certain temperature range.

## Organization of the Thesis

Chapter 2 contains a brief review of literature concerning the Marshall Mix design and the use of the tensile splitting test as a test method for measuring tension indirectly in asphalt concrete specimens.

Chapter 3 discusses some of the physical properties of asphalt that can be used to explain the differences in behavior of the asphalt concrete specimens.

The Marshall Mix Design used in the preparation of the specimens, properties of the asphalt and aggregates, the testing program and a description of the instruments used in the test are outlined in Chapter 4.

Chapter 5 contains the results of the Marshall Mix Design review as well as the data resulting from the testing program.

Chapter 6 discusses the results outlined in the previous chapter.

Chapter 7 contains the conclusions and recommendations for further study in this research program.

Data sheets, typical calculations, theoretical aspects and detailed procedures are contained in the appendixes.

# CHAPTER 2

# MIX DESIGNS AND TENSILE SPLITTING TEST

## Introduction

This chapter proposes to review some of the recent research on asphalt mixture design that is pertinent to the investigation of asphalt mixture designs that forms part of this thesis. Conventional methods of mixture design assume that the most critical period in pavement life occurs during the summer months at the higher temperatures, and 140°F has been selected as the temperature at which the strength parameters are measured. At low temperatures, conventional methods of mixture design do not consider any strength characteristic that can be related to the causes of transverse cracking. It is proposed to use the tensile splitting test as a means of investigating the low temperature behavior of asphalt mixtures. The physical properties of asphalt cement and their relation to the low temperature performance of asphalt mixtures will be reviewed in a succeeding chapter.

## Literature Review

The most essential properties required in an asphalt mixture design are stability and durability. Conventional

mix design methods used to attain these properties have been developed and are based on selecting the best gradation of aggregate at an asphalt content to give a desirable value for the air voids in the compacted mixture (Voids Method); or the amount of asphalt required to coat the surfaces of the aggregate particles (Surface Area Method).

Campen (1957) showed that air voids in aggregates having the same gradation can be very different. Continuing this research, it has also been shown by Campen (1959) that durability and stability in a mixture were obtained when the aggregate contained enough voids to permit the addition of sufficient asphalt to provide a film thickness of at least 6 microns on each aggregate particle without filling all the voids in the aggregate. In other research, Goode (1965), suggested that high air void content, thin asphalt films and air permeability of the mix could be related to hardening. Although test results did not establish the importance of air permeability on hardening, they did indicate that use of a combined factor of a ratio of air void content to bitumen index is satisfactory for comparing resistance to asphalt hardening of different mixes regardless of the gradation of the aggregate blend. It was also indicated that the Marshall method of mix design might be improved by the substitution of a maximum voids-bitumen index ratio for the presently used maximum air void content.

These additional criteria will be used to provide a basis for a further check on the properties of the Marshall mixture designs that have been used in cracked and uncracked sections of highways and previously reported by Shields (1964). Shields concluded in his report that the occurrence of cracking could not be related to any property of the Marshall mixture designs.

## Cracking of Asphalt Pavements

The preceding discussion has been related to the stability and durability of the mixes and makes no reference to the transverse cracking of pavements with which we are primarily concerned. Vallerga (1955), and Hveem (1958), have published extensive reviews on the types of cracking in asphalt pavements but do not refer specifically to the transverse cracking of pavements.

A review of literature shows that Rader (1935) investigated the physical properties of asphalt mixes at low temperatures and tried to relate the results to the cracking and deterioration of asphalt pavements. Rader assumed the cracking of asphalt pavements at low temperatures occurred in the same way as did cracking of concrete pavements, that is, by excessive tensile stresses caused by contraction.

In more recent years, the Highway Research Board Committee on Design of Bituminous Paving Mixtures (1955) has emphasized the importance of cracking but no specific design recommendations were suggested to eliminate the problem. Shields (1964) has discussed the possible mechanisms of cracking and a research program sponsored by the Alberta Cooperative Highway Research Program is in progress to investigate the effects of subgrade, sub-base and different sources of asphalt. Mollard (1964) and Skarsgard (1964) have reported similar types of transverse cracking in the Province of Saskatchewan. Domaschuk (1964) reported the results of a laboratory program in which the cause of cracking was attributed to volume changes in the asphalt concrete itself and suggested that cracking took place at a critical temperature of -4° F. In the discussion following this report, Young mentioned the cracking problem in the Province of Manitoba and discussed the placing of a series of transverse cuts in newly laid asphalt pavement in an attempt to reduce cracking. The result of a very short observation period indicates that transverse cuts at spacings of 500 feet or less might control the thermal cracking of asphalt pavements.

Different approaches to the phenomenon of transverse cracking have been investigated by other researchers. A recent publication by Monismith (1965) has described the application of the principles of viscoelastic behavior to

predict the thermal stresses which may be induced in asphalt concrete slabs at low temperatures. The predicted thermal stresses in the asphalt concrete slabs were substantiated by a laboratory program and it was concluded that temperature changes, particularly in the range below 0°F, might contribute to the cracking of asphalt concrete pavements.

Zube and Cechetini (1965) have shown that highly absorptive aggregates used in mixes will result in similar transverse cracking patterns. Of particular interest is the fact that cracking occurs during the warmest hours of the day. The mechanism of cracking is described as follows:

> "Asphalt concrete specimens fabricated from absorptive aggregates and exposed to normal atmospheric conditions absorb moisture from the air which may cause considerable expansion. During the warmest hours of the day, these expanded specimens contract causing transverse, or block cracking."

Apart from the effects of the highly absorptive aggregates as reported by Zube and Cechetini (1965), the results of the preceding paragraphs indicate that the problem of cracking is widespread in the western Canadian provinces and northern United States which are subjected

to long periods at lower temperatures during the winter months.

Tension Testing of Asphalt Concrete Mixture

Asphalt concrete pavements crack when the thermal stresses induced in the pavement by low temperatures exceed the breaking strength of the mixture. This is a failure in tension, and in spite of this, conventional mix design methods do not contain a standard test method for tension testing of asphalt concrete mixes. Past research on the tensile properties of asphalt concrete mixes have been concerned with a specific investigation and in most cases rectangular shaped specimens were used which are very different from the cylindrical shaped specimen formed for conventional mix design methods. The nearest approach to a tension test is the cohesiometer test as used in the Hveem method of mix design. This test is performed at 140° F and because of its empirical nature cannot be related to any fundamental property of the asphalt concrete mixture, which is essential in any tension test.

The cylindrical shaped specimens specified by most conventional mix design methods are not readily adaptable to tension testing because of their shape. Hewitt (1965) has experimented with different shaped molds which are of the same thickness and contain the same volume of asphalt mixture; but are of different cross sectional area as the

specimen prepared by Marshall Mix Design method. Since the Marshall Method of mix design is used by the Alberta Department of Highways, it would be desirable to have a test method capable of measuring a strength parameter on specimens formed in the laboratory or on those that have been removed from the road surface.

The tensile splitting test and its application as a tension test for concrete cylinders has been described by Thaulow (1957) and should be adaptable for use with asphalt specimens at low temperatures. Since its acceptance as a concrete test, researchers have improved on the method and investigated its applications for other materials. Wright (1955) improved testing procedures by suggesting that the load be applied to the concrete cylinder through packing strips of relatively soft material and limited the width of these strips to approximately one-twelfth the specimen diameter. The use of loading strips reduces the infinite compressive forces that develop under a line load. The test has also been used with stabilized and natural materials. Frydman (1964), used the test on soils which become slightly flattened under load and was able to show that the deformation in the sample does not invalidate the use of the formula for calculating tensile strength from an indirect tension test. Thompson (1965) used the test to study the characteristics of lime-stabilized soil mixtures and was able to relate these results to the compressive strength of the mixture.

In the field of asphalt concrete, Livneh and Shklarsky (1962), have used the splitting test as a means of measuring strength of prismatic shaped specimens and consider the test to be very useful for the design and control of asphalt mixes. In other research, and more closely related to the investigation, Stephens and Breen (1965) have used the tensile splitting test to evaluate the tensile strength of cylindrical specimens from 0° F to 40° F. Their results indicate that ultimate tensile strength of the mixture increased with decreasing temperature down to 20° F. From 20° F to 0° F there is only a slight decrease in the ultimate tensile strength. From the limited information published regarding Stephens' and Breen's work, the application of the tensile splitting test as a method of measuring the tensile strength of asphalt mixtures appears warranted. This investigation, in addition to the ultimate tensile strength as measured by Stephens and Breen, will attempt to measure strain and tensile strength of the specimens until failure occurs. The tensile strength and strain characteristics will then be used to assess the differences in behavior of asphalt mixtures using asphalts from different sources.

Further details regarding the distribution of stress and the theoretical considerations of the tensile splitting test are contained in Appendix A.

# CHAPTER 3

# PHYSICAL PROPERTIES OF ASPHALT CEMENTS AND MIXTURES

This Chapter will discuss the properties of asphalt cements and mixtures that influence the low temperature behavior of asphalt concrete mixtures. The properties to be discussed are temperature-susceptibility, stiffness modulus and the glass transition temperature.

## Temperature Susceptibility

Asphalt is considered to be a thermoplastic material because its viscosity is sensitive to temperature change. This sensitivity of asphalt to changes in temperature is important in its applications to highway construction. Before an asphalt cement can be mixed with aggregate, it must be heated to higher temperatures to ensure that the asphalt cement is in the required viscosity range to adequately coat the aggregate particles. Further, it may also be useful to know the viscosity of asphalt cement at the temperature of rolling, or at the temperatures encountered during its service life. The extent to which the viscosity of an asphalt changes with the temperature has been termed the temperature-susceptibility of the asphalt.

A number of methods are available for defining the temperature versus viscosity relationships of asphalt and several of these will be discussed. The first of these

are the Penetration Ratio and Penetration Index which are based on consistency tests. According to Monismith (1961), the Penetration Ratio is determined from the standard penetration test at 77° F and 39.2° F and is calculated from this relationship:

$$\text{Penetration Ratio} = \frac{\text{Pen. at } 39.2^\circ\text{ F}}{\text{Pen. at } 77^\circ\text{ F}} \begin{matrix} (200 \text{ gr. } 60 \text{ sec}) \\ (100 \text{ gr. } 5 \text{ sec}) \end{matrix}$$

By selecting a minimum value for the Penetration Ratio the temperature susceptibility of the asphalts can be controlled.

The Penetration Index (P.I.) is based on the measurement of consistency at two temperatures; penetration at 77° F and the ring and ball softening point temperature. A nomogram is used to determine the P.I. from these values. Monismith (1961) lists the range of values for Penetration Index and most asphalts fall in this range. The P.I. for paving grade asphalts varies from + 2 to - 2; an asphalt with a P.I. of + 2 is considered to be highly temperature susceptible, conversely, an asphalt with a P.I. of - 2 shows little susceptibility to temperature change.

Another technique for measuring temperature susceptibility, based on fundamental viscosity measurements, is to specify the slope of the temperature versus viscosity

relationships for asphalts when viscosity is plotted in absolute units. Two methods are suggested by Monismith (1961) and both are merely attempts to define a straight line relationship between temperature and viscosity. The first of these is the Walther formula which states

$$\log \log \left[ \text{viscosity} + \text{constant} \right] = \log \text{absolute temperature}^{\circ} \text{ Kelvin} + \text{constant}$$

The second method is the Waterman formula:

$$\log \text{viscosity} = \frac{\text{constant}}{(\text{Absolute Temperature})^{x}} + \text{constant}$$

where x has a value of 3.5 to 4 for most asphalts. In both methods a comparison of temperature susceptibility is made on the basis of the numerical value of the slope of the line. The steeper the slope, the more temperature susceptible the asphalt.

Figure 2 illustrates the relationship for 3 asphalts based on the Walther formula. Penetration Ratios of the asphalts are also shown on the figure.

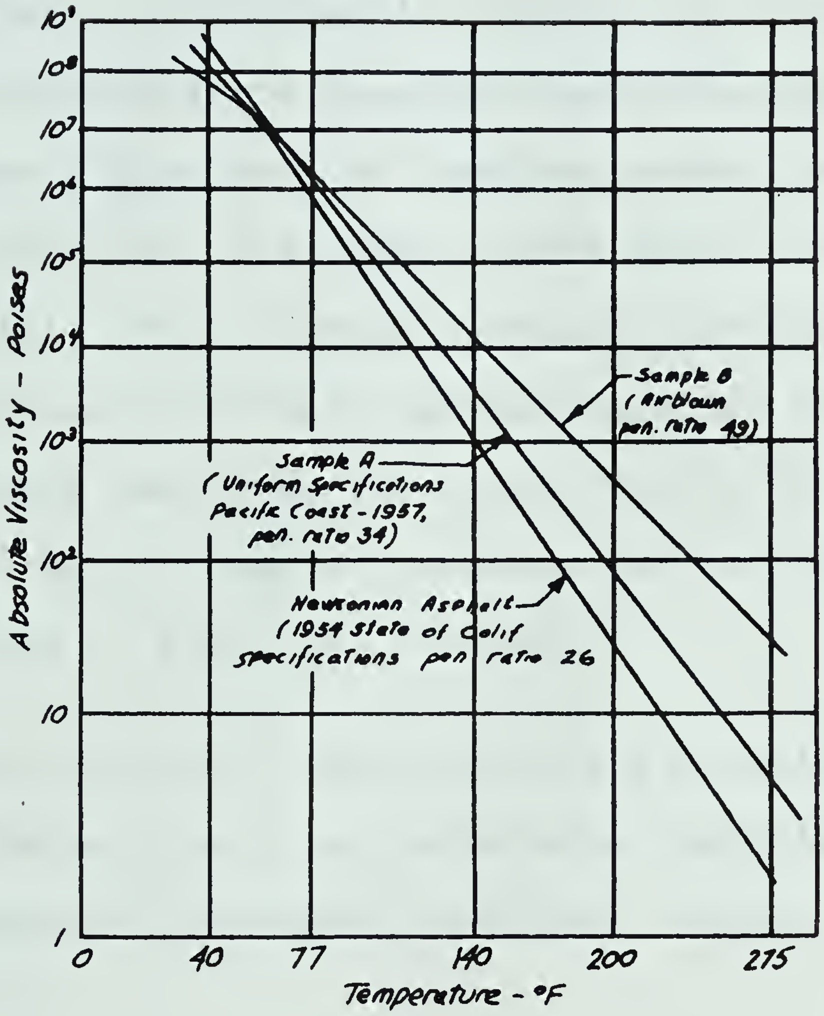



FIGURE 2. Temperature vs viscosity relationships for asphalt cements [log log (viscosity + constant) vs log absolute temperature]. Viscosity determined at shear rate of $5 \times 10^{-2}$ sec $^{-1}$ (Monismith 1961)

According to Traxler (1961), the degree of temperature susceptibility of an asphalt can be related to its crude source and chemical properties. Properties of asphalts can be changed during processing by air blowing and to a certain extent is beneficial because it results in a less temperature susceptible material.

## Stiffness Modulus

Asphalt is termed a thermoplastic material since its consistency or viscosity varies with the temperature. It is also termed a viscoelastic material as its stress strain characteristics are dependent upon the rate of loading. Under a slow rate of loading, asphalt behaves as a viscous material, and under a fast rate, as an elastic material. With so many variables involved, it would be desirable to devise a system enabling the user to deduce quickly the mechanical properties of a given asphalt in the whole range of temperatures and rates of loading that are of practical interest.

Van der Poel (1954) devised such a relationship and defined the modulus of a viscoelastic material as a time and temperature dependent stiffness modulus given by the equation

$$(S)_{t,T.} = \frac{\sigma}{\varepsilon}\, t,T.$$

for any particular combination of rate of loading t and temperature T, where S is stiffness in pounds per square inch, $\sigma$ is stress in pounds per square inch and $\varepsilon$ is strain in inches per inch. Time of loading can vary between a dynamic type with short loading times or a static type with much longer loading times and can be expressed as a function of loading time and frequency. Van der Poel has constructed a nomograph using this relationship between

dynamic and static loading expressed as a function of loading time and frequency, Penetration Index, and difference in temperature between Ring and Ball Softening Point and selected test temperature.

The stiffness concept in conjunction with the nomogram can provide a method to place the stress strain relationship on a numerical basis at a particular time and temperature and can be used to predict behavior of asphalt cement at other times and temperatures.

The values of stiffness obtained from the nomograph refer to asphalt cements alone and do not apply to asphalt mixtures. Recently, Heukelom and Klomp (1964), have examined the stiffness of mixtures in detail and have suggested the following expression using the same factors as developed by Van der Poel:

$$\frac{S_{mix}}{S_{asphalt}} = \left[ \left(1 + \frac{2.5}{n}\right) \left(\frac{C_v}{1-C_v}\right) \right]^n$$

where $S_{mix}$ is stiffness of mixture in pounds per square inch

$S_{asphalt}$ is stiffness of asphalt cement in pounds per square inch

$C_v$ is the volume concentration of aggregate

and is equal to $\dfrac{\text{Volume of Aggregate}}{\text{Volume of Aggregate + Asphalt}}$

$$n = 0.83 \log \frac{4 \times 10^5}{S_{asphalt}}$$

Heukelom and Klomp suggest that this expression can be used for mixtures with $C_V$ values between 0.7 and 0.9 and air void contents found in well compacted mixtures. When applied to this thesis, the expression may be useful in checking the values of stiffness as calculated from the laboratory testing program.

## Glass Transition Temperature

The properties of asphalt concrete mixes change in accordance with the changes in the properties of the asphalt binder. Viscosity is a fundamental property of asphalt cement and various instruments are used for measuring viscosity at temperatures above 39.2° F. At low temperatures, viscosity measurements cannot be made because of the extreme hardness or brittleness of the material. Thus, conventional test methods are not available for measuring properties of asphalts at these low temperatures.

In the field of polymer science, researchers use the Glass Transition Temperature ($T_G$) to distinguish between the brittle and plastic states. The similarity between the behavior of asphalts and the behavior of viscoelastic polymers has been observed and Barrall (1964) suggests that asphalts can be treated as a variety of viscoelastic, low molecular weight, polymers. As defined by Ferry (1961), "the $T_G$ of any amorphous substance,

whether polymeric or not, may be defined as the point where the thermal expansion coefficient undergoes a discontinuity." Barrall further describes the $T_G$ as "the temperature below which there is insufficient room between the molecules for them to rotate or move except for in place vibration. This would mean that there is very little tendency for the material to flow, and the elastic modulus is very high."

When applied to asphalt cements, it may be postulated that the $T_G$ will have the following effects upon the behavior of asphalt cements. At temperatures below the $T_G$ the material is considered to be elastic, but because of its brittle nature it is only capable of withstanding very small strains before failure. Above the $T_G$, the behavior is plastic and the degree of plastic behavior is dependent upon the temperature and rate of loading.

## Summary

Several physical properties that may affect the low temperature behavior of asphalt cements and mixtures have been discussed. Other properties such as volume change or thermal expansion, may also influence cracking frequency but this approach is beyond the scope of this thesis and will not be discussed.

# CHAPTER 4

## PREPARATION OF SPECIMENS AND TESTING PROGRAM

This Chapter discusses the preparation of specimens, the testing program, and laboratory procedure developed to utilize the Tensile Splitting Test for measuring the stress strain characteristics of asphalt concrete specimens at low temperatures.

### PREPARATION OF SPECIMENS

#### Background

The recent work by Sharan (1965) on the tensile properties of asphalt cements, recovered from cracked and uncracked sections of the highway provided a basis for the planning of this thesis. Sharan's studies showed that Stiffness Modulus of asphalt cements removed from cracked sections of highway was always greater than that removed from the uncracked sections. In recommending further studies on asphalt mixtures, Sharan raised the question of whether higher or lower values of stiffness can also be related to the cracked and uncracked sections of highway. It was impractical to establish a program for determining the stiffness of asphalt mixtures from the same section of highway used by

Sharan. Another approach would be to assume that aggregate type itself is not an important factor in the cracking phenomena and the real reasons for cracking can be attributed to the quality of the asphalt cement. Following this reasoning, the basic ingredients required for a testing program would be aggregate and asphalt cement. The only restriction in the selection of the ingredients would be that the aggregate is to be used in asphalt mixtures for future paving projects and the asphalt cements must come from sources that have been associated with high and low cracking frequencies in the past.

Selection of Asphalt Cement

It was necessary to select asphalt cements that have been associated with different cracking behaviors in the past. Samples of asphalt cements had been obtained directly from the projects using them in 1964 and were available for this testing program. Among those available were asphalt cements from two sources which have been associated with different cracking behavior, however these were not available in the same penetration grade. This difference in penetration grade is not considered to be an important factor because both asphalt cements are of approximately the same consistency or hardness at the lower temperatures. The two asphalt cements selected are referred to as Asphalt A and Asphalt B; Asphalt A has been used in pavements which have consistently shown higher cracking frequences than Asphalt B.

Some of the physical properties of Asphalt A and Asphalt B are shown in Table I.

Table I

<u>Physical Properties of Asphalts (a)</u>

| | | Asphalt A | Asphalt B | |
|---|---|---|---|---|
| Penetration Grade | | 200-300 | 150-200 | |
| Penetration at 77° F | | 203 | 153 | |
| | 39.2°F (b) | 54 | 54 | |
| | 32°F (b) | 27 | 33 | |
| Penetration Ratio (c) | | 26 % | 35 % | |
| Ring and Ball Softening Point °F | | 102.4 | 107.2 | |
| Viscosity Data (d) | | | | |
| Temp °F | Shear Rate | | | Units-poise |
| 140° | $10^{-3}$ | 12.10 | 6.30 | $\times 10^2$ |
| | $10^{-1}$ | 4.80 | 6.50 | |
| 77 | $10^{-3}$ | 3.50 | 3.40 | $\times 10^5$ |
| | $10^{-1}$ | 1.45 | 3.08 | |
| 39.2 | $10^{-3}$ | 8.50 | 15.3 | $\times 10^7$ |
| | $10^{-1}$ | 7.80 | 10.6 | |

(a) All data supplied by Research Council of Alberta, Highway Research Division

(b) 200 grams, 60 seconds

(c) Penetration Ratio = $\dfrac{\text{Pen at } 39.2°\text{F}}{\text{Pen at } 77°\text{F}} \times 100$

(d) By Shell Sliding Plate Viscometer

## Aggregate

An aggregate suitable for our requirements was available from a stockpile located at the Ponoka No. 3 Gravel Pit. The gradation of the aggregate, shown in Table II are the mean results of sieve analysis obtained during the seven weeks the crusher had been in operation. The aggregate appeared to be well graded and within the specification limits as prescribed by the Department of Highways without the addition of blending materials.

Table II

Gradation of Aggregate

| U. S. Standard Sieve Series No. | % Passing |
|---|---|
| 3/4 | 100 |
| 4 | 52 |
| 10 | 35 |
| 40 | 14 |
| 200 | 7.4 |

Table III shows some additional physical characteristics of the aggregate.

Table III

Physical Characteristics of Aggregate

| | |
|---|---|
| Apparent Specific Gravity | 2.684 |
| Effective Specific Gravity | 2.617 |
| Bulk Specific Gravity | 2.563 |
| Absorption | 0.83 |

## Marshall Mix Design

The Marshall Mixture Design was prepared by the Department of Highways for a section of Highway No. 2 north of Ponoka which is to be included in the 1966 paving program. The mixture design used the aggregates previously described and a 150-200 penetration grade of asphalt cement as the binder. It is emphasized that the asphalt cement used in this design is not related in any manner to the cracking of pavements but is used in accordance with standard procedure within the Department of Highways.

The procedure followed by the Department of Highways in the preparation of the mixture design and cylindrical specimens was in accordance with ASTM Designation D 1559, "Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus", except that an automatic tamper was used to compact the specimens. Sixty-five blows were applied to each face of the specimen.

The summary sheet for the Marshall Mixture Design and graphs showing the relationships between percent asphalt content versus stability, percent air voids, percent voids filled, density, and flow values are both contained in Appendix C.

Excerpts from the summary sheet, listing some of the more important properties of the mixture design at asphalt contents above and below that suggested for use in the field mixture are contained in Table IV.

Table IV

Properties of Marshall Mixture Design (a)

| | | | Suggested Field Design | |
|---|---|---|---|---|
| Asphalt Content (b) | 5.0 | 6.0 | 6.7 | 7.0 |
| Density lbs/cu. ft. | 139.7 | 141.0 | 141.7 | 141.9 |
| Stability lbs. | 1050 | 1160 | 1310 | 1385 |
| Flow 0.01 inches | 9.0 | 9.0 | 9.5 | 10.5 |
| % Voids Mineral Aggregate | 16.8 | 16.9 | 17.0 | 17.0 |
| % Voids Filled with Asphalt | 52.0 | 64.5 | 72.0 | 75.0 |
| % Air Voids Total Mix | 8.1 | 6.0 | 4.8 | 4.3 |

(a) All data interpolated from curves drawn and attached in Appendix C.

(b) Asphalt Content in pounds per 100 lbs. of aggregate.

# TESTING PROGRAM AND LABORATORY PROCEDURE

## Preliminary Testing

Some investigation was necessary before the testing program was undertaken to eliminate some of the variables and establish test procedures. The tensile splitting test, originally developed for concrete testing has been used for other elastic or brittle materials. For asphalt materials, elastic behavior is dependent upon temperature and rate of applied strain. It was not possible to investigate fully the influence of rate of deformation and temperature becasue of the large number of specimens involved. Therefore, it was necessary to select a range of low temperatures and a rate of deformation such that the behavior of the asphalt specimens approached that of an elastic material.

Several different methods of measuring deformation across the diameter of the specimen were considered. The destructive nature of the tensile splitting test precluded the use of bonded or unbonded electrical strain gauges. The Demec Strain Gauge, adapted for a two inch gauge length was tried, but the difficulty of holding the instrument on a vertical surface, introduced erratic results, and its use was not seriously considered. Linear Variable Differential Transformers (LVDT) and recording apparatus were also assessed but the range of deformation covered by the specimen during

the test was unknown. It was concluded that deformation in this testing program could best be measured by optical means. A Tuckerman Optical Strain Gauge was available and the use of this instrument was investigated. The range of deformation that can be measured with the Tuckerman Strain Gauge is limited to approximately 0.012 inches. The deformation developed in the specimen is dependent upon the gauge length, rate of deformation and the temperature at which the test is carried out. The gauge length was set at one inch to ensure that the maximum stress occurring along the vertical diameter of the specimen was reflected in the deformation measurement. Further details regarding the determination of the gauge length are included in Appendix A. The rate at which specimen was deformed was set at 0.045 inches per minute with test temperature between -10° F and + 20° F and was determined on a trial and error basis.

Of the strain measuring devices that were available the Tuckerman Strain Gauge was the most suitable and was therefore used for this research project.

## Testing Program

The testing program was set up to determine the difference in tensile stress and induced strain characteristics in the cylindrical specimens of asphalt mixtures formed from two asphalts obtained from different crude

sources and of different penetration grade. Variables in the testing program were asphalt type, asphalt content in percent, and test temperatures. Variables are shown in Table V.

Table V

Testing Program

| | |
|---|---|
| Asphalt Type | A, B |
| Testing Temperatures | - 10°F, 0°F, 10°F, 20°F |
| Asphalt Content | 5% 6% 7% |
| Rate of Deformation (constant) | .045" per min |

Three specimens for each test condition were to be tested in the program. Considering the two different asphalt sourses, 4 temperatures and 3 asphalt contents which resulted in 24 test conditions, a total of 72 specimens were required for the complete testing program. It was realized that 3 specimens for each test condition were insufficient for a statistical study of the results but the large number of specimens necessary, limited the size of the program. No allowance was made for poor specimens, accidents during the test, or unusual results and thus an additional 38 specimens were prepared to verify the results for each test condition.

## Formation of Specimens for Testing Program

Cylindrical specimens for the proposed testing program were formed from mixtures of the selected aggregates and

Asphalt A and Asphalt B. Aggregate gradation was kept identical with the gradation used by the Department of Highways in their original Marshall Mix Design by screening the aggregate into size fractions and then recombining the size fractions in the amounts necessary to produce the desired gradation. The asphalt contents of the specimens were set at 5,6 and 7 percent of one hundred pounds of aggregate, to ensure that the results of this testing program would cover the percent asphalt content eventually used in the field mixture.

The procedure used in the formation of these specimens adhered to ASTM Designation, D 1559 "Resistance to Plastic Flow of Bituminous Mixtures" using the Marshall Apparatus except as follows:

(1) Compacting the Specimen

An automatic tamper was used to compact the specimens and sixty-five blows were applied to each face of the specimen.

(2) Mixing and Compacting Temperatures

Since Asphalt A and Asphalt B are of different penetration grades, 200-300 and 150-200 respectively, it was necessary to ensure that the mixing and compaction temperatures were related to the temperature-viscosity curves for each asphalt.

For mixing, aggregates at 310° F were mixed with asphalt at 275° F, at which temperature the viscosity of both asphalts were within the range as suggested by the Asphalt Handbook (1962) for adequate coating of the aggregate particles. For compaction, the mix using Asphalt A was compacted in the 245° F - 250° F range and the mix using Asphalt B was compacted in the 250° F - 255° F range; at which temperatures the viscosities of Asphalt A and B were approximately equal.

## Preparation and Testing of Specimens

Each specimen was inspected for obvious flaws, gauge marks were attached, and specimens were cooled to the desired temperatures before the test was carried out at room temperatures. A detailed procedure of all the steps performed from the initial inspection of the specimen and including the running of the test is contained in Appendix A.

## Examination of Specimen After Testing

Each specimen was inspected after testing to observe any unusual occurrences and other characteristics which could be used to explain the results. Slipping of the strain gauge during the test, fracture of the specimen on the opposite side of the specimen to that on which the

strain gauge was attached, the presence of large voids or excessive crushing of the specimen under the wooden blocks were considered as unusual occurrences. The type of fracture, whether plastic or brittle; failures along the asphalt aggregate interface; size and types of aggregate fractured; even or uneven distribution of aggregate fractures over the cross sectional area; were all observed and eventually used as a basis for assessing the validity of the test method.

## Application of Load and Calculation of Stress

A Tinius Olsen Hydraulic Press was used to load the specimens. Readings were taken at every 200 kilogram increment of load and the induced strain readings in the specimen were recorded. The maximum tensile stress across the specimen can be calculated by the use of Frocht's (1948) formula:

$$\sigma_x = \frac{2P}{\pi d t}$$

where $\sigma_x$ is tensile stress in psi

$P$ is applied load in pounds

$t$ is thickness of specimen in inches

$d$ is diameter of specimen in inches

In addition to the stress and strain readings that were taken for every 200 kilogram increment of load, the loading dial reading for every minute was recorded as well as the total time to failure.

Measurement of Strain

Through the use of the Tuckerman Strain Gauge the induced strain in the specimen was measured for every 200 kilogram increment of applied load to failure. Strain is calculated through the use of the following formula appearing in the Instruction Manual No. 750 (1958):

$$S = \frac{F\ A\ L\ R}{E \times 1000}$$

where S is strain in inches per inch

F is calibration factor for 0.2" lozenge = 1.006

A is calibration factor for collimator = 1.003

L is lozenge size in inches

R is elapsed reading from start of test in complete divisions

E is gauge length in inches

Further details on the Tuckerman Strain Gauge are contained in Appendix A.

CHAPTER 5

# RESULTS

This Chapter contains results pertaining to the Marshall Mix Design study which comprised part of this thesis, followed by laboratory data derived from tensile splitting tests performed on asphalt specimens.

## Results From Marshall Mix Design Study

The bases of this mix design study are six selected areas within a fifty mile radius of the City of Edmonton that have shown different transverse cracking characteristics. The areas studied were chosen by the ACHRP and reported by Shields (1964).

Table VI shows mix properties derived from laboratory mix design data and from samples that have been removed from the test sections. Most of the data used in Table VI has been taken from Table A of the report mentioned above, but has been rearranged such that results are presented in ascending order of cracking frequency.

The cracking frequency as presented in Table VI was rated in half mile intervals, extrapolated and expressed as cracks per mile. It is pointed out that the number of cracks is an average value and may not represent the maximum

# TABLE VI

## PROPERTIES OF MARSHALL MIXTURE DESIGN

| CRACKS PER MILE AVERAGE | ASPHALT SUPPLIER | ROUTE | MILE TO MILE | MIX DESIGN NO. | STABILITY IN POUNDS | FLOW IN 0.01 IN. | ASPHALT CONTENT % | | UNIT WEIGHT LBS. CU. FT. | | AIR VOIDS % | | % V.M.A. | | % VOIDS FILLED | | SURFACE AREA SQ.FT. LB. | | FILM THICKNESS IN MICRONS | | BITUMEN INDEX X $10^{-3}$ | | VOID-BITUMEN INDEX RATIO | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | DES. | EXT. | DES. | EXT. | DES. | EXT. | DES. | EXT. | DES. | EXT. | DES. | EXT. | DES. | EXT. | DES. | EXT. | DES. | EXT. |
| 0 | 7 | 2–G4 | 219.5–220.0 | 58 | 1220 | 9.5 | 5.7 | 5.9 | 147.2 | 145.0 | 3.4 | 8.2 | 14.8 | 17.3 | 77.0 | 52.6 | 30.6 | 24.8 | 8.5 | 11.15 | 1.3 | 2.3 | 1.9 | 3.6 |
| 0 | 3 | 12–B2 | 47.4–63.7 | 65 | 960 | 10.0 | 6.5 | 6.6 | 144.4 | 146.0 | 4.6 | 7.0 | 16.5 | 16.7 | 72.0 | 58.1 | 22.5 | 25.4 | 13.2 | 11.9 | 2.7 | 2.4 | 1.7 | 2.7 |
| 0 | 10 | 21–A1 | 26.13–26.34 | 145 | 1305 | 10.0 | 5.5 | 4.7 | 146.9 | 143.4 | 3.4 | 10.2 | 13.2 | 20.0 | 75.5 | 48.3 | 25.5 | 28.7 | 10.3 | 7.2 | 2.1 | 1.5 | 1.6 | 7.0 |
| 0 | 5 | 57–A | 14.55–16.55 | 191 | 1540 | 10.0 | 5.8 | 5.9 | 145.2 | 141.7 | 3.6 | 10.4 | 15.0 | 18.3 | 76.0 | 43.0 | 31.00 | 34.6 | 8.4 | 8.1 | 1.7 | 1.7 | 2.1 | 6.3 |
| 5 | 5 | 34–A | 58.0–58.5 | 112 | 1330 | 8.0 | 5.2 | 6.7 | 146.5 | 143.1 | 3.5 | 8.6 | 14.2 | 18.7 | 74.8 | 54.0 | 33.0 | 24.9 | 7.3 | 12.8 | 1.5 | 2.6 | 2.4 | 3.3 |
| 7 | 1 | 2D–13 | 17.31–24.97 | 286 | 1600 | 12.0 | 6.1 | 5.7 | 140.4 | 141.5 | 6.1 | 10.3 | 17.0 | 16.1 | 64.0 | 36.1 | 48.4 | 40.1 | 5.1 | 5.8 | 1.1 | 1.0 | 6.0 | 10.2 |
| 8 | 3 | 2D–11 | 0.0–8.13 | 273 | 1390 | 12.0 | 5.5 | 5.4 | 141.9 | 142.9 | 5.6 | 10.6 | 16.0 | 15.4 | 65.8 | 31.2 | 31.8 | 35.4 | 7.8 | 6.5 | 1.6 | 1.3 | 3.1 | 8.0 |
| 72 | 10 | 21–A1 | 29.0–29.38 | 145 | 1305 | 10.0 | 5.4 | 5.4 | 146.9 | 144.4 | 3.4 | 4.6 | 13.4 | 16.1 | 75.5 | 71.4 | 25.5 | 28.6 | 10.3 | 7.5 | 2.1 | 1.6 | 1.6 | 3.2 |
| 75 | 4 | 12–B2 | 63.7–77.4 | 109 | 995 | 9.0 | 6.8 | 6.7 | 142.9 | 139.4 | 4.3 | 10.3 | 16.6 | 20.5 | 74.5 | 49.8 | 32.8 | 31.5 | 9.2 | 9.6 | 1.9 | 2.0 | 2.3 | 5.3 |
| 30 | 7 | 2–G4 | 225.0–225.5 | 58 | 1220 | 9.5 | 5.7 | 6.2 | 147.2 | 144.9 | 3.4 | 8.1 | 14.8 | 17.6 | 77.0 | 54.0 | 30.6 | 23.7 | 7.8 | 12.2 | 1.6 | 2.5 | 2.1 | 3.2 |
| 39 | 1 | 2D–12 | 8.13–17.31 | 273 | 1390 | 12.0 | 5.5 | 5.8 | 141.9 | 144.6 | 5.6 | 8.1 | 16.0 | 14.6 | 65.8 | 44.5 | 31.8 | 32.8 | 7.7 | 7.6 | 1.6 | 1.6 | 3.1 | 5.3 |
| 158 | 1 | 34–A | 45.0–45.5 | 101 | 1540 | 12.0 | 6.1 | 5.9 | 145.0 | 145.2 | 4.0 | 7.3 | 15.5 | 16.1 | 74.5 | 54.7 | 39.00 | 26.8 | 7.0 | 10.1 | 1.4 | 2.1 | 2.8 | 3.5 |
| 229 | 10 | 21–A1 | 24.21–26.13 | 145 | 1305 | 10.0 | 5.4 | 5.2 | 146.9 | 143.5 | 3.4 | 6.7 | 13.4 | 17.1 | 75.5 | 62.9 | 25.5 | 32.0 | 10.3 | 7.2 | 2.1 | 1.5 | 1.6 | 4.6 |
| 348 | 1 | 57–A | 12.55–14.54 | 148 | 1310 | 9.0 | 5.9 | 5.3 | 144.5 | 138.5 | 3.4 | 12.4 | 14.5 | 19.2 | 75.6 | 35.4 | 35.5 | 38.7 | 7.4 | 6.0 | 1.5 | 7.2 | 2.2 | 10.2 |

DES. --- MARSHALL MIX DESIGN

EXT. --- RECOVERED SAMPLE

number of cracks appearing in a mile of the highway rated. The asphalt supplier refers to the refinery source and for use in presentation has been numbered 1-10 by the ACHRP. Marshall Mix Design information was obtained from the Department of Highways (DOH). Extraction results were also obtained from ACHRP after an extensive sampling program had been carried out.

The remainder of the data in the table represents the contribution of the author. Surface Area Coefficients are based on those developed by the California Department of Highways (1942).

Sample calculations for surface area and film thickness, bitumen index, and voids-bitumen index ratio are included in Appendix B.

Results of Tensile Splitting Tests

A summary of the results of the Testing Program are shown in Table VII. Table VII is set up to show test number and the corresponding maximum stress, strain and total elapsed time at failure. Tests not used in computing the mean value of stress and strain are designated with an X. The reasons for rejecting tests will be discussed in the next chapter. Stiffness modulus is calculated from the mean values of stress and strain.

# TABLE VII

## RESULTS OF TENSILE SPLITTING TESTS

**− 10°F**

| ASPHALT CONTENT | ASPHALT A TEST NO. | ASPHALT A TIME [1] | ASPHALT A STRESS [2] | ASPHALT A STRAIN [3] | ASPHALT B TEST NO. | ASPHALT B TIME [1] | ASPHALT B STRESS [2] | ASPHALT B STRAIN [3] |
|---|---|---|---|---|---|---|---|---|
| 5% | 1 | | 400 | 21.0 | 37 | | 414 | 20.0 |
| | 2 X | | 287 | 44.6 | 38 | | 344 | 29.8 |
| | 3 X | | 303 | 50.0 | 39 | 6-30 | 399 | 23.2 |
| | 73 | | 392 | 30.4 | 115 | 7-00 | 415 | 24.2 |
| | 74 | | 446 | 20.4 | | | | |
| | 78 | | 496 | 20.0 | | | | |
| | | MEAN | 431 | 22.3 | | MEAN | 391 | 24.2 |
| | | STIFFNESS | 13.3 [4] | | | STIFFNESS | 16.3 | |
| 6% | 13 X | 5-00 | 378 | 45.2 | [illegible] | 5-15 | 307 | 96.0 |
| | 14 | 6-00 | 453 | 22.0 | 50 | 6-15 | 429 | 17.8 |
| | 15 | 6-00 | 498 | 16.0 | 51 | 6-15 | 406 | 14.0 |
| | 33 X | 7-45 | 440 | 7.0 | 116 | 7-10 | 440 | 17.2 |
| | 34 | 3-30 | 496 | 12.0 | 102 X | 3-30 | 445 | 29.0 |
| | 109 | 3-00 | 525 | 22.0 | | | | |
| | | MEAN | 493 | 18.0 | | MEAN | 425 | 16.3 |
| | | STIFFNESS | 27.2 | | | STIFFNESS | 26.1 | |
| 7% | 25 X | 4-00 | 432 | 47.3 | 61 | | 450 | 20.0 |
| | 26 X | | 457 | | 62 | | 503 | 18.0 |
| | 27 X | | 457 | | 63 | | 453 | 23.2 |
| | 35 X | | 468 | 37.6 | 103 X | 9-15 | 364 | 47.6 |
| | 36 | | 541 | 17.2 | | | | |
| | 87 | 3-00 | 578 | 20.0 | | | | |
| | 90 | 3-30 | 513 | 27.5 | | | | |
| | | MEAN | 541 | 21.6 | | MEAN | 467 | 20.4 |
| | | STIFFNESS | 26.2 | | | STIFFNESS | 22.9 | |

**0°F**

| ASPHALT CONTENT | ASPHALT A TEST NO. | ASPHALT A TIME [1] | ASPHALT A STRESS [2] | ASPHALT A STRAIN [3] | ASPHALT B TEST NO. | ASPHALT B TIME [1] | ASPHALT B STRESS [2] | ASPHALT B STRAIN [3] |
|---|---|---|---|---|---|---|---|---|
| 5% | 4 | 6-30 | 337 | 33.6 | 40 | 7-00 | 302 | 42.0 |
| | 5 | 5-30 | 364 | 24.4 | 41 | 6-15 | 386 | 25.0 |
| | 6 | 5-00 | 424 | 21.2 | 42 X | 6-00 | 271 | 80.0 |
| | | | | | 96 | 6-15 | 366 | 36.6 |
| | | | | | 112 | 6-00 | 344 | 36.0 |
| | | MEAN | 375 | 26.5 | | MEAN | 344 | 35.0 |
| | | STIFFNESS | 14.1 | | | STIFFNESS | 9.8 | |
| 6% | 16 | 5-30 | 440 | 19.4 | 52 | 5-45 | 347 | 35.0 |
| | 17 | 5-00 | 440 | 20.4 | 53 | 6-45 | 350 | 23.8 |
| | 18 | 5-30 | 414 | 24.6 | 54 | 6-15 | 367 | 25.2 |
| | | | | | 98 | 6-00 | 414 | 29.6 |
| | | MEAN | 431 | 21.5 | | MEAN | 375 | 28.4 |
| | | STIFFNESS | 20.0 | | | STIFFNESS | 13.6 | |
| 7% | 28 | 5-00 | 446 | 27.0 | 64 X | 6-30 | 385 | 87.2 |
| | 29 X | 6-30 | 358 | 65.2 | 65 X | 6-00 | 300 | 94.8 |
| | 30 | 6-00 | 482 | 23.4 | 66 | 6-00 | 435 | 32.0 |
| | 111 | 6-30 | 441 | 29.6 | 104 | 6-00 | 396 | 26.4 |
| | | | | | 105 | 6-15 | 418 | 30.0 |
| | | | | | 119 | 7-15 | 406 | 28.0 |
| | | MEAN | 456 | 26.6 | | MEAN | 414 | 29.1 |
| | | STIFFNESS | 17.0 | | | STIFFNESS | 14.2 | |

**10°F**

| ASPHALT CONTENT | ASPHALT A TEST NO. | ASPHALT A TIME [1] | ASPHALT A STRESS [2] | ASPHALT A STRAIN [3] | ASPHALT B TEST NO. | ASPHALT B TIME [1] | ASPHALT B STRESS [2] | ASPHALT B STRAIN [3] |
|---|---|---|---|---|---|---|---|---|
| 5% | 7 X | | 277 | 70.6 | 43 | 5-45 | 292 | 71.4 |
| | 8 X | 6-30 | 363 | 21.2 | 44 | 5-30 | 246 | 68.4 |
| | 9 | 5-45 | 277 | 29.0 | 45 | 5-30 | 314 | 58.2 |
| | 75 | 8-15 | 343 | 31.8 | 114 | 6-30 | 316 | 50.0 |
| | 76 | 9-00 | 290 | 37.6 | | | | |
| | | MEAN | 303 | 32.8 | | MEAN | 292 | 82.0 |
| | | STIFFNESS | 9.2 | | | STIFFNESS | 4.8 | |
| 6% | 19 | 6-15 | 402 | 32.0 | 55 X | 6-15 | 108 | 93.6 |
| | 20 | | 263 | | 56 | 5-45 | 325 | 57.8 |
| | 21 | 4-30 | 386 | 34.0 | 57 | 6-00 | 302 | 55.2 |
| | 80 | 7-30 | 375 | 29.2 | 117 | 7-30 | 283 | 49.2 |
| | 82 | 7-45 | 391 | 31.8 | 118 | 6-45 | 376 | 60.0 |
| | | MEAN | 388 | 31.6 | | MEAN | 345 | 55.5 |
| | | STIFFNESS | 12.2 | | | STIFFNESS | 6.2 | |
| 7% | 31 | 5-00 | 411 | 17.4 | 67 | 5-30 | 324 | 69.8 |
| | 32 | 6-00 | 372 | 37.4 | 68 X | | 216 | |
| | 33 | 6-00 | 275 | 54.0 | 89 | 6-45 | 370 | 79.4 |
| | 88 | 8-45 | 380 | 30.6 | 106 | 9-40 | 273 | 74.6 |
| | | | | | 120 | 7-45 | 290 | 84.0 |
| | | MEAN | 360 | 34.9 | | MEAN | 314 | 76.9 |
| | | STIFFNESS | 10.3 | | | STIFFNESS | 4.1 | |

**20°F**

| ASPHALT CONTENT | ASPHALT A TEST NO. | ASPHALT A TIME [1] | ASPHALT A STRESS [2] | ASPHALT A STRAIN [3] | ASPHALT B TEST NO. | ASPHALT B TIME [1] | ASPHALT B STRESS [2] | ASPHALT B STRAIN [3] |
|---|---|---|---|---|---|---|---|---|
| 5% | 10 | 7-15 | 226 | 69.4 | 46 | 4-40 | 247 | 71.8 |
| | 11 | 6-10 | 219 | 49.2 | 47 | 5-00 | 195 | 72.0 |
| | 12 | 8-00 | 262 | 65.2 | 48 | 4-30 | 162 | 67.0 |
| | 77 | 7-45 | 209 | 57.6 | 113 | 6-00 | 202 | 90.0 |
| | | MEAN | 229 | 60.3 | | MEAN | 201 | 75.2 |
| | | STIFFNESS | 3.8 | | | STIFFNESS | 2.7 | |
| 6% | 22 | 6-00 | 289 | 75.8 | 58 | 5-30 | 225 | 86.0 |
| | 23 | 6-00 | 276 | 68.8 | 59 | 5-15 | 210 | 75.3 |
| | 24 | 5-15 | 278 | 53.2 | 60 | 5-45 | 226 | 81.4 |
| | 110 | 7-15 | 337 | 59.4 | 100 | 5-45 | 234 | 76.0 |
| | | MEAN | 295 | 64.3 | | MEAN | 231 | 96.7 |
| | | STIFFNESS | 4.6 | | | STIFFNESS | 2.3 | |
| 7% | 34 | 6-30 | 325 | 80.8 | 70 | 5-30 | 273 | 117.6 |
| | 35 | 6-00 | 305 | 76.0 | 71 X | | 260 | |
| | 36 | 6-15 | 381 | 72.8 | 72 | 5-15 | 275 | 116.4 |
| | 89 | 6-30 | 269 | 79.2 | 108 | 7-00 | 175 | 117.2 |
| | | | | | 121 | 6-50 | 209 | 119.2 |
| | | MEAN | 320 | 77.2 | | MEAN | 233 | 117.8 |
| | | STIFFNESS | 4.1 | | | STIFFNESS | 2.0 | |

[1] TIME TO FAILURE IN MINUTES AND SECONDS

[2] STRESS IN POUNDS PER SQUARE INCH AT FAILURE

[3] STRAIN IN INCHES PER INCH X $10^{-4}$ AT FAILURE

[4] STIFFNESS IN PSI X $10^4$ AT FAILURE

Figure 3 shows a typical set of stress strain curves for test condition 6 % asphalt A at - 10° F. Data sheets for each test are attached in Appendix C.

Sample calculations are included in Appendix A for maximum tensile stress, maximum strain and the Stiffness Modulus.

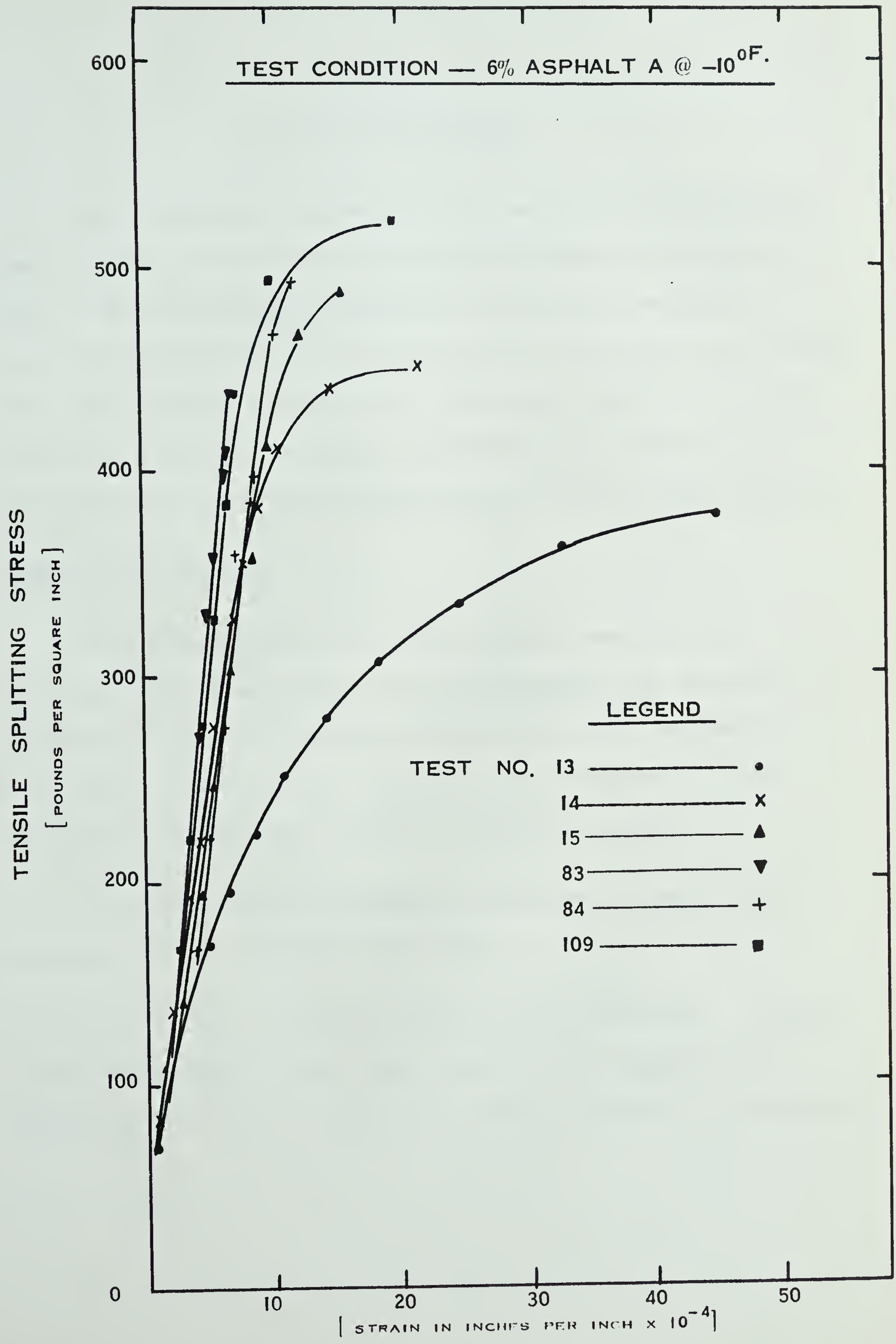



FIGURE 3 STRESS VERSUS STRAIN

# CHAPTER 6

## DISCUSSION OF RESULTS

This Chapter studies the mix designs that have been used in the selected study areas and attempts to relate some of these physical properties of the mix designs to the cracking problem. The validity of the tensile splitting test, the sources of error and laboratory results are presented in a manner that will illustrate the differences in the behavior of Asphalt A and Asphalt B at low temperatures.

### Marshall Mix Design

The properties of the mix designs used in the selected study areas have been investigated and reported by Shields (1964). It was concluded that the incidence of cracking could not be related to any property of the mix using conventional methods of mixture design.

For this thesis, several additional criteria for checking the mix designs were used.

The first to be discussed is film thickness. Campen (1959) has suggested that each particle of aggregate be coated with a film of asphalt at least 6 microns in thickness

as a minimum criterion for design. Campen's suggested criterion is not concerned with cracking of pavements but rather with stability and durability of asphalt mixtures. The purpose in investigating Campen's criterion is to determine whether thin asphalt films are associated with the greatest cracking frequencies. By referring to Table VI, and using the extracted samples (Ext.), it can be seen that:

> Four sections showed no cracking and film thickness varied between 7.2 and 11.9 microns.
>
> Three sections had cracking frequencies of 5 to 8 cracks per mile and film thickness varied from 5.8 to 12.8 microns.
>
> Four sections had cracking frequencies of 72 to 89 cracks per mile and film thickness varied from 75 to 12.2 microns.
>
> Three sections had cracking frequencies of 158 to 348 cracks per mile and film thickness varied from 6.0 to 10.1 microns.

Although the two highest cracking frequencies (229 and 348) are associated with film thicknesses of 7.2 and 6.0 respectively, values of film thickness in the same range are also associated with low cracking frequencies. This data does not indicate any trend between film thickness and cracking frequency, and on the basis of these results, the 6 micron criterion for film thickness can hardly be considered as a minimum limiting value in mix design for the prevention or reduction of cracking. Thus it may be

considered that the incidence of cracking cannot be related to the film thickness of asphalt on the aggregate particles.

In another approach Goode (1965), has suggested a maximum voids-bitumen index ratio (VB) of 4 in place of the maximum air void content presently specified in the Marshall method of mix design as a means of controlling hardness of asphalt pavements. If hardening of asphalts was a major factor to be considered in the cracking of pavements, then it would be expected that the oldest pavements would exhibit the largest frequency of cracking and be related to the highest VB ratio. By referring to Table VI, it is observed that VB ratio for the extracted samples varies from 2.7 to 10.2; and maximum frequency of cracking occurs in a pavement 4 years old with the highest VB ratio. But other comparisons do not show any relationship and it is therefore considered that the incidence of cracking cannot be related to the age of the pavement and its VB ratio.

## TENSILE SPLITTING TEST

### Validity of the Tensile Splitting Test

In the transverse cracking of pavements, it has never been established just where cracking takes place: do the cracks pass through the aggregate particles or are they located in the asphalt phase? The tensile splitting test

has been selected to measure the stresses and strains that exist in the laboratory specimens at the time of failure. The relation these laboratory measured stresses and strains have with field cracking cannot be established at this time.

The first part of this discussion will be concerned with the validity of the tensile splitting test as a test method.

In the tensile splitting test, fracture of the aggregate particles was obtained as shown in Figure 4 and 5. Figure 4 shows typical split specimens for the complete range of test temperatures. Figure 5 is a composite photo of specimens for every test condition. By inspecting the split specimens in the laboratory, several differences have been noted through the range of test temperatures. At the lower test temperatures of - 10°, 0° and 10° F, it was observed that:

(a) Aggregate fractures occurred over the whole cross sectional area

(b) All sizes and types of aggregate were fractured

(c) Separations along the asphalt aggregate interface were negligible

At the highest test temperature of 20° F, it was observed that:

(a) Aggregate fractures were not evenly distributed over the whole cross sectional area.

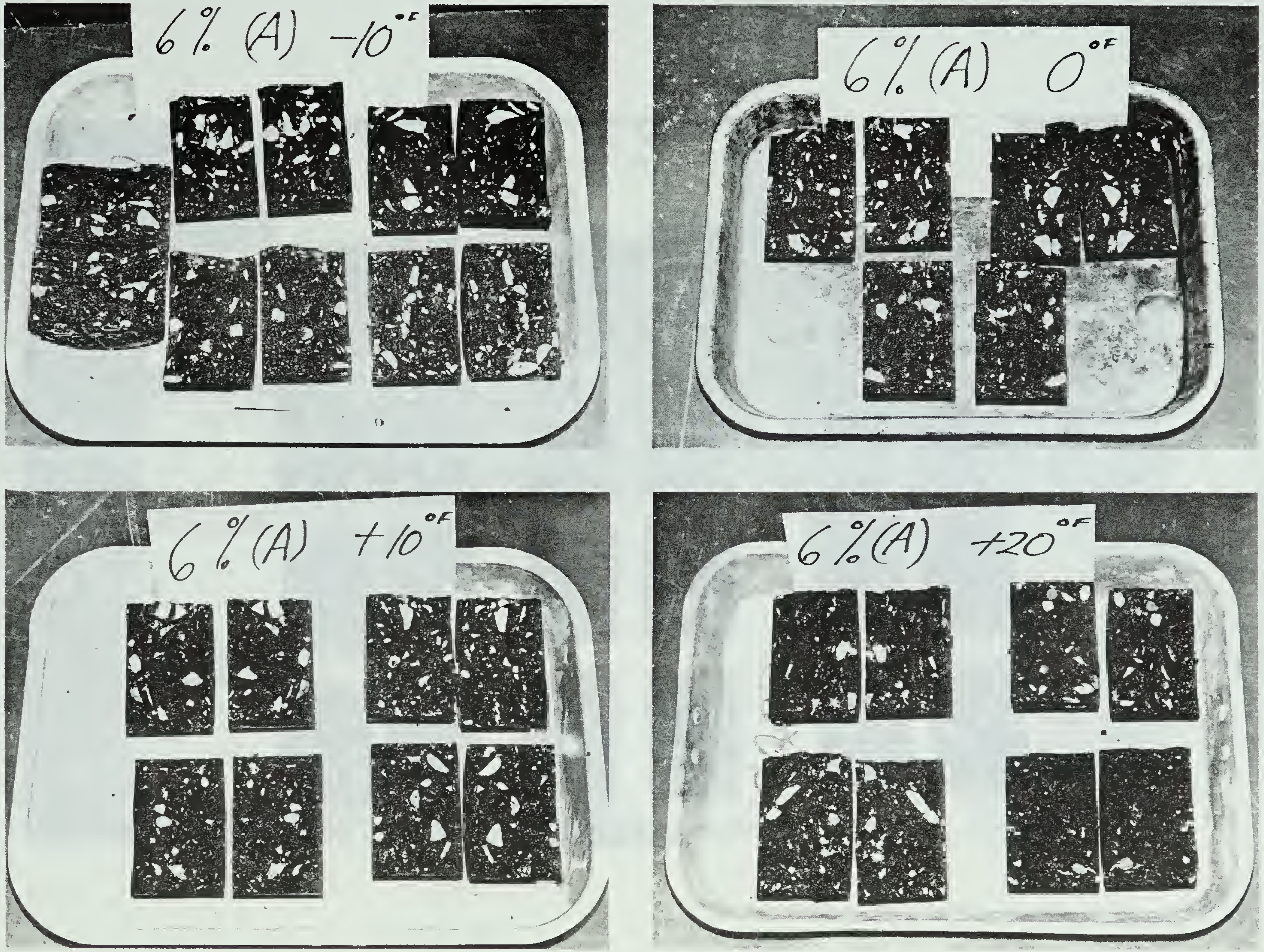



FIGURE 4 SPLIT SPECIMENS FROM 6% ASPHALT A AT -10°, 0°, 10°, 20°F.

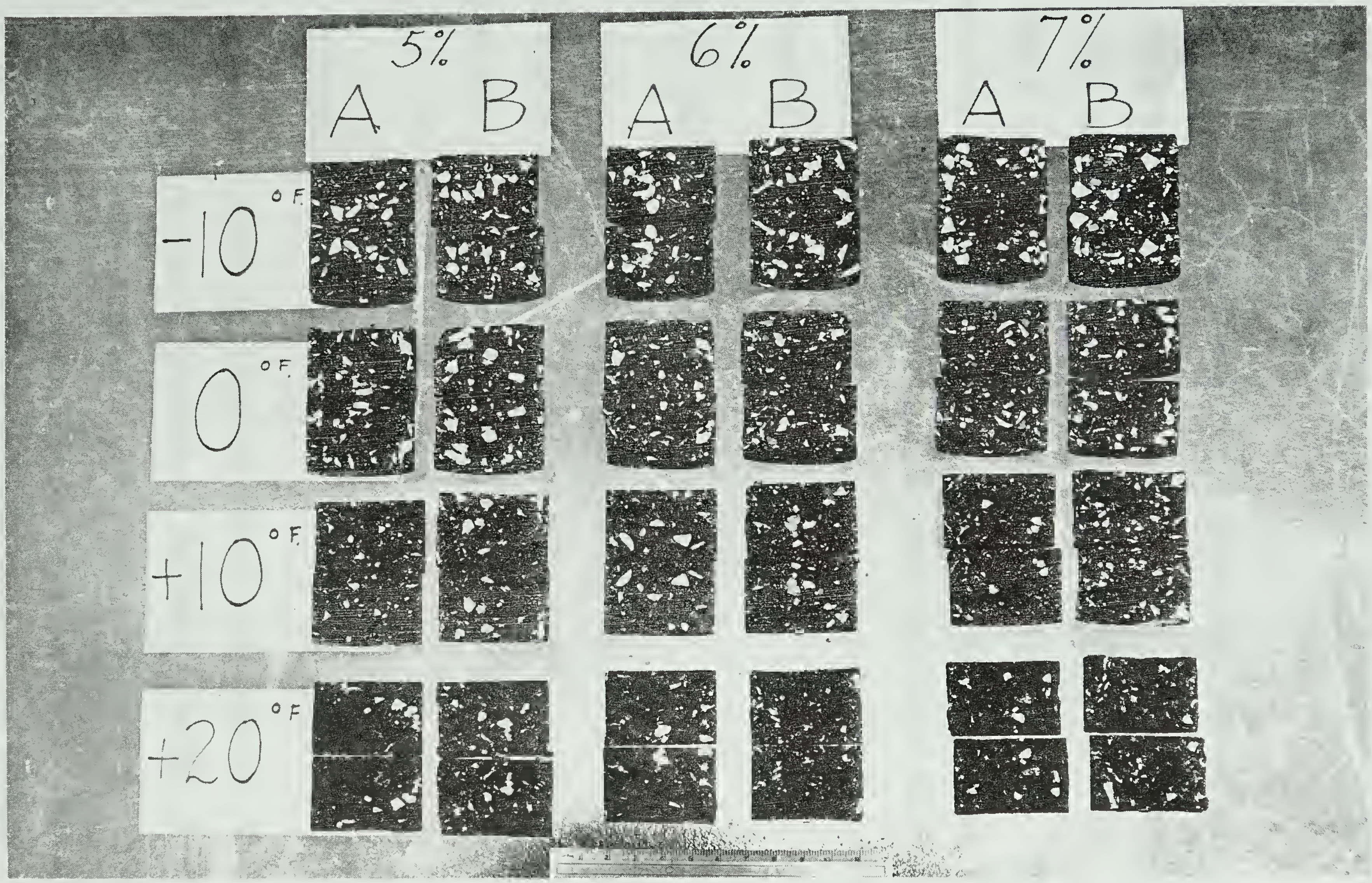



FIGURE 5 SPLIT SPECIMENS FROM ALL TEST CONDITIONS

(b) Most of the fractures occurred in the larger particles of the soft aggregates.

(c) Numerous separations appeared along the asphalt aggregate interface.

The interference from these observations is that, at low temperatures, the asphalt binder is strong enough to hold aggregates in position until the aggregate is pulled apart by the induced tensile stress. At the higher temperatures, the binder is not strong enough, except for the softer pieces of aggregate, to hold the aggregate in position and failure occurs as a result of fractures along the aggregate asphalt interface.

Sources of Error

Errors resulting from test conditions and procedures undoubtedly affected the stress-strain characteristics of the asphalts. The chief sources of error can be attributed to:

(a) Temperature

Specimens were chilled in the frost room at the test temperature for a period of 24 hours before testing at room temperatures. The time required for setting up was kept at 2 minutes or less and the time of testing varied from 5 to 9 minutes. No attempt

was made to control the temperature of the specimen during the testing and the errors resulting from the lack of temperature control were not assessed.

(b) Rate of Deformation and Rate of Loading

The rate of deformation for all specimens was kept constant at 0.045 inches per minute. It was expected that specimens tested under the same conditions when subjected to the same rate of deformation would fail in approximately the same length of time. This was not the case and time to failure varied from 5 to 9 minutes in some test conditions which meant that specimens were subjected to variable rates of loading. At the lower test temperatures the variation in time to failure should not affect the strength properties because the asphalt was assumed to behave as an elastic material. At the higher test temperatures, it would be expected, that longer time to failure would result in larger strains and reduced tensile stresses in the specimens. However, length of time to failure does not appear to be related in any way to the tensile properties of the specimens in each test condition. The manner of approach for checking the differences in time to failure was to compare measured stiffness at failure, with calculated stiffness at failure using the empirical formula

developed by Heukelom and Klomp (1964). This empirical formula was based on direct tension tests and a uniaxial state of stress. However in the tensile splitting test, a biaxial state of stress exists and the effects of Poisson's ratio cannot be ignored.

By presuming a biaxial state of stress, the strain measured across the vertical diameter of the specimen will be the result of compressive stresses in the Y direction and tensile stresses in the X direction, and according to Boyd and Folk (1950) is represented as follows:

$$\varepsilon_x = \frac{1}{E}(\sigma_x - \mu\sigma_y)$$

where $\varepsilon_x$ is strain in X direction

$E$ is modulus of elasticity

$\sigma_x$ is tensile splitting stress and equal to $2P/\pi dt$

$\sigma_y$ is compressive stress and equal to $6P/\pi dt$

$\mu$ is Poisson's ratio and assumed value is 0.3

By substitution,

$$\varepsilon_x = \frac{1}{E}\left[\frac{2P}{\pi dt} - (0.3)\left(-\frac{6P}{\pi dt}\right)\right]$$

$$= \frac{1}{E}\left[\frac{2P}{\pi dt} + \frac{1.8P}{\pi dt}\right] = \frac{P}{E\pi dt}(2 + 1.8)$$

it is shown that the strain due to the tensile stress is approximately half the total value of $\varepsilon_x$.

Table VIII shows the stiffness of the mix measured in the laboratory using the strain values due to tensile stresses in the X direction, compared to the stiffness of mix as calculated from Heukelom and Klomp's empirical formula discussed on Page 22. The ratio of stiffness calculated to stiffness measured varies from 6.5 : 1 to 2.2 : 1.

Table VIII

Comparison of Stiffness

| Test No | Stiffness Of Mix (Measured) PSI | Stiffness Of Mix (Measured) Kg/cm$^2$ | Time To Failure Min. | Time To Failure Sec. | Stiffness Of Asphalt Calculated Kg/cm$^2$ | Stiffness Of Mix Calculated Kg/cm$^2$ | Ratio S $\frac{Calc.}{Meas.}$ |
|---|---|---|---|---|---|---|---|
| 14 | 410,000 | 28,500 | 6 | | 80,000 | 188,000 | 6.5:1 |
| 15 | 622,000 | 43,200 | 6 | | 80,000 | 188,000 | 4.3:1 |
| 84 | 812,000 | 57,000 | 8 | | 50,000 | 125,000 | 2.2:1 |
| 109 | 476,000 | 33,000 | 8 | 30 | 50,000 | 125,000 | 3.8:1 |

Another reason presented for the large discrepancies is in the use of Van der Poel's nomograph. The stiffness of the asphalt cement as determined from the nomograph is much greater than the 25000 - 30000 kg/cm$^2$ values indicated by Heukelom and Klomp. The reasons must be in the fact that the loading time used referred to the time to failure for the specimens of asphalt mixture. Further, the nomograph was constructed for use with loads of the dynamic or static type and the loading used in our testing program was in effect, a variable rate of loading.

(c) Measurement of Strain

Using the Tuckerman Strain Gauge, in many of the tests, it was not possible to observe precisely the deformation at maximum load. Near maximum load deformation increased very rapidly for small increases in load which resulted in the reflected image moving too quickly to record the reading at failure.

The presence of errors resulting from the test procedures is acknowledged. It is assumed that these errors will not be too detrimental to the main scope of this thesis; that is of devising a test method for determining the low temperature characteristics and differences in behavior of two asphalt types.

## Behavior of Specimens During Test

The appearance of the split specimens can also be related to their behavior during the test. At low temperatures, the specimen behaves as an elastic material with a straight line relationship between stress and strain almost up to failure. This was followed, in some tests by a short range of plastic deformation to the failure point. As the temperature is increased, the range in which the specimen acts as an elastic material is decreased and the range in which the specimen acts as a plastic material is increased.

## Selecting or Rejecting Test Results

The preparation of asphalt specimens is dependent upon too many factors to ensure that identical specimens would have identical values for stability, flow or density. These variations in stability, flow and density in specimens formed by the Marshall Method have been reported by Corbett (1956), Vokac (1962) and Hode Keyser (1963). It was therefore expected that variations would be present in the stress

and strain properties of the specimens that were to be tested in this program.

Upon completion of the 72 specimens comprising the first testing program, the percent variation from the mean stress and strain values for each test condition were calculated and found to be as high as 30 % for stress and over 200 % for some of the strain measurements. From the initial results it was obvious that additional testing would be required and a method devised of selecting or rejecting test results.

The small number of samples in each test condition precluded the selection or rejection of tests by statistical methods. The method used, was to reject those tests because of unusual occurrences during the test or because the shape of the stress strain curve did not conform with the remainder in the test condition. Unusual behavior consisted of slippage of the strain gauge during the test or cracking of the specimen on the side opposite to that on which the gauge was positioned and six tests were rejected for this reason. The reader is referred to Figure 3 and this phenomenon is shown by the behavior of Test No. 83. Fourteen tests were rejected because of the shapes of the stress strain curve which were totally unrelated to the remaining curves of the test condition. This type of behavior is shown by Test No. 13.

The remainder of the tests were used and mean values of stress and strain were calculated. Variation from the mean was about 15 % for stress and up to 33 % for strain. It was considered that the 15 % variation is acceptable and stress will not be discussed further. The variations in the strain values up to 33 % appear to be associated with the lower test temperatures, - 10° F, 0° F and 10° F. The behavior of the specimens during test has been discussed in the previous section. In Figure 3 the elastic range of the specimens is quite consistent up to the point where plastic deformation occurs and it is observed that much of the strain at failure results from a small change in induced tensile stress.

Another method was considered for determining strain at failure. By projecting the straight line portion of the stress strain curve upward until it intersects the ordinate of the stress at failure, a strain value is obtained which is based on the assumption of complete elastic behavior to failure. This method could not be applied at the 20° temperature because of the plastic behavior of the specimens and was therefore not adopted for the lower temperatures.

# GRAPHICAL PRESENTATION OF DATA

Results obtained in the testing program have been plotted in figures that follow to best illustrate the differences between the two asphalts and the relations that exist between strain, stress, Stiffness Modulus, and Temperature. Other figures are presented that are used to discuss the application of these tests to the design of asphalt mixtures.

## Strain Versus Temperature

Before discussing the behavior of the asphalts an explanation of the term strain as it applies to this testing program is given. The strain at failure, in inches per inch, measured across the vertical diameter resulted from the biaxial state of stresses that existed in the specimen during the test. Since Poisson's ratio was not determined, it was not possible to assess the proportion of the measured strain that was due to the tensile splitting stress. The strain at failure used in Figure 6 represents the average strain measured between gauge marks one inch apart on the specimen. Magnitude of this strain is estimated at 90 % of the true value of the strain resulting from the maximum stress in the center of the specimen at failure. This is explained further in Appendix A under Determination of Gauge Length.

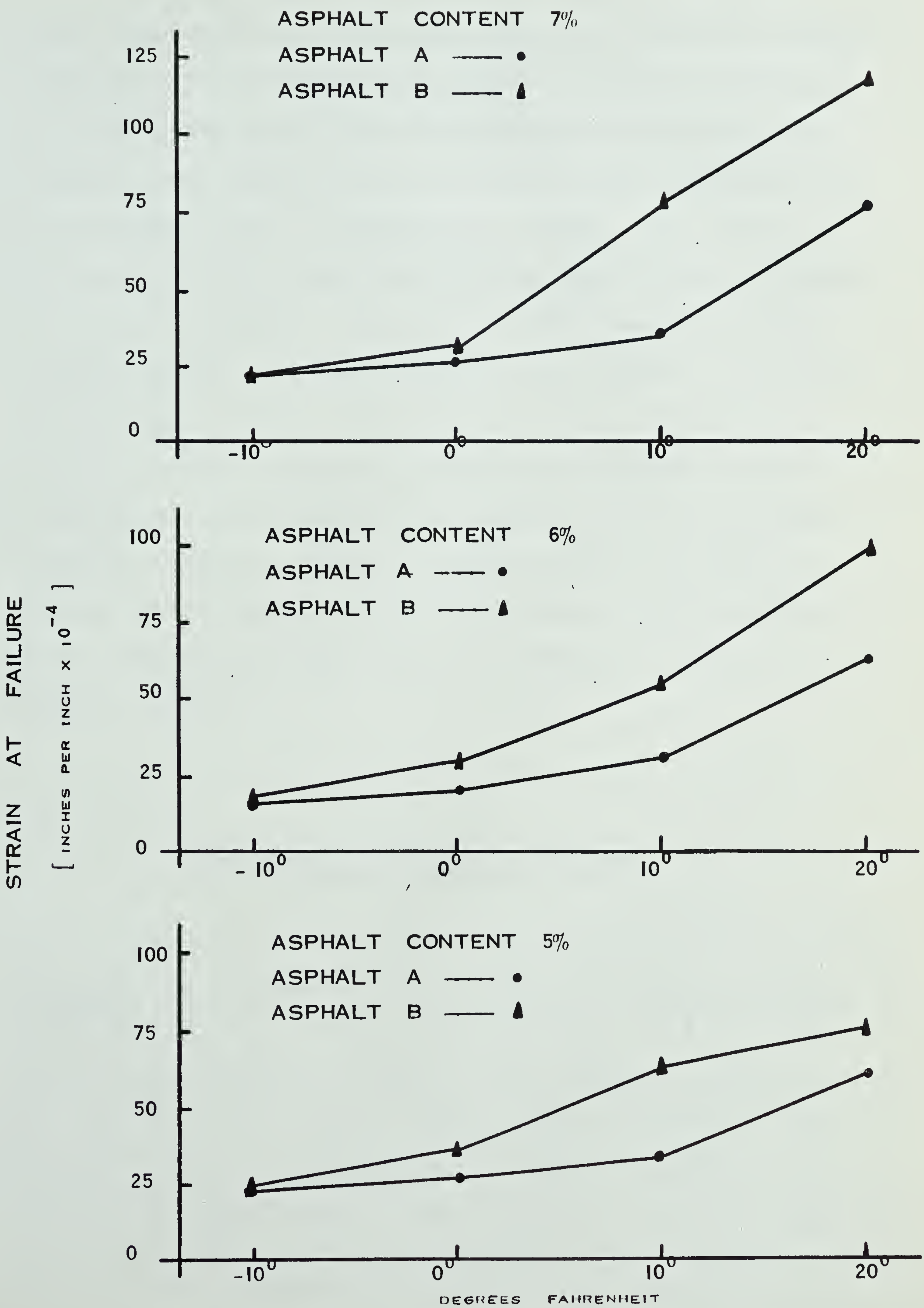



FIGURE 6 STRAIN AT FAILURE VERSUS TEMPERATURE

Figure 6 shows the strain at failure versus the test temperatures for Asphalts A and B at different asphalt contents. For both asphalts, strain at failure is smaller at lower temperatures and increases as temperature rises. Asphalt A and Asphalt B do not show the same behavior over the complete range of temperature tested. At - 10° F, both asphalts show approximately the same strain at failure but vary with asphalt content. At 0° F, Asphalt B shows slightly greater strain at failure than Asphalt A but vary with asphalt content. At 10° F and 20° F, strain at failure for both asphalts increases but Asphalt B shows considerably more strain than Asphalt A at the different asphalt contents. Strain at failure, and the change in strain between Asphalt A and B are listed in Table IX for test temperatures - 10° F and 20° F and 5 %, 6 % and 7 % asphalt content.

Table IX

Comparison of Strain at Failure

| Asphalt Content % | - 10°F | | + 20°F | | Change in Strain | |
|---|---|---|---|---|---|---|
| | A (%) | B (%) | A (%) | B (%) | A % | B % |
| 5 | .23 | .24 | .60 | .75 | .37 | .51 |
| 6 | .16 | .16 | .64 | .99 | .48 | .83 |
| 7 | .22 | .20 | .77 | 1.18 | .55 | .98 |

From Table IX it can be seen that Asphalt B is able to withstand more strain to failure than Asphalt A between - 10° F and 20° F; and this characteristic of Asphalt B to withstand more strain than A increases as asphalt content increases.

The shape of the strain versus temperature curve also indicates that Asphalt A reaches low strain values at higher temperatures than Asphalt B. At 7 % asphalt content, this temperature occurs at 10° F for Asphalt A and at 0° F for Asphalt B. At the lower asphalt contents (5 % and 6 %), the same phenomenon can be observed but the changes in the slopes of the curves are not as definite as shown for the 7 % asphalt content.

Strength Versus Temperature

Stress discussed in this thesis refers to the tensile splitting stress as calculated from the relationship:

$$\sigma_x = 2P/\pi dt$$

The theoretical assumptions outlined in Appendix A presuppose an elastic material and one in which compressive strength is at least three times tensile strength. The extent to which test conditions satisfy these assumptions cannot be assessed on the basis of the compressive strength-tensile strength ratio because compressive strengths are not available. At the test temperatures of - 10°, 0° and 10° F,

it was observed from the stress strain curves that specimens acted as elastic materials during the test except for a short range of plastic behavior near failure. At 20° F, the shape of the stress strain curves indicated predominantely plastic behavior throughout the tests.

Figure 7 shows the tensile strength at failure versus temperature for Asphalt A and B at different asphalt contents. The graphs show that tensile strength varies with temperatures and asphalt content: at low temperatures, strength is high and at high temperatures strength is low, the higher the asphalt content the higher the strength. The difference in behavior between Asphalt A and B is readily apparent; Asphalt A exhibits more tensile strength than Asphalt B under the same test conditions.

The shape of the strength at failure versus temperature graphs does not indicate any specific property of the asphalt. For the temperatures being considered, a linear relationship may be said to exist between stress at failure and temperature.

## Stiffness Modulus Versus Temperature

Stress and strain at failure plots versus temperature may be combined to show the Stiffness Modulus at

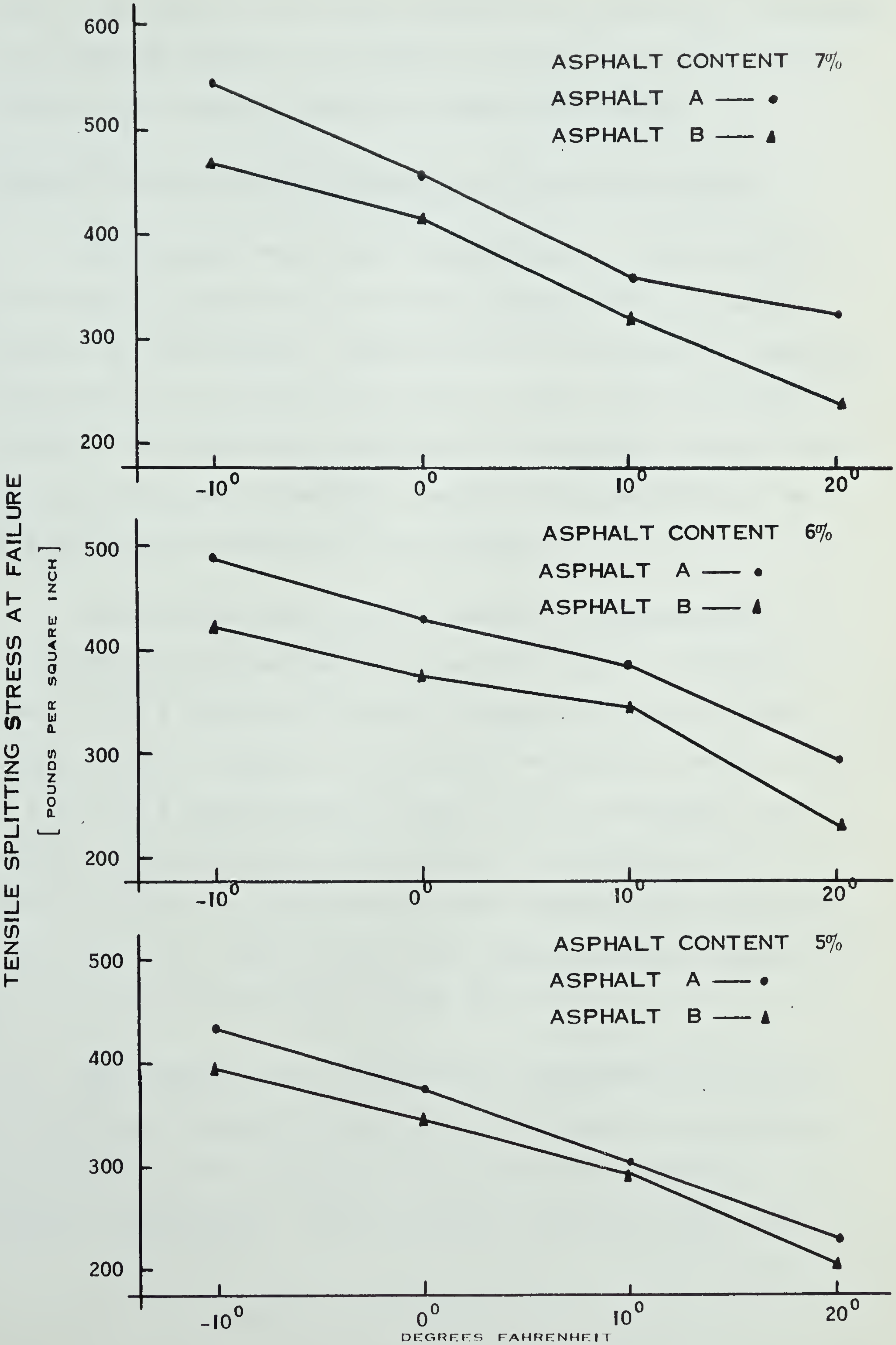



FIGURE 7 STRESS AT FAILURE VERSUS TEMPERATURE

failure versus temperature as in Figure 8. This figure shows a decrease in Stiffness Modulus as temperature increases. The Stiffness Modulus for Asphalt A is greater than that of Asphalt B through the complete temperature range.

Reasons for Difference in Behavior of Asphalt A and B

The reasons which may be presented to describe the difference in behavior of the two asphalts are, no doubt, related to the chemical composition of the asphalts, however, this approach is beyond the scope of this thesis and was not studied. One approach, would be to investigate temperature susceptibility and determine whether this property can be related to the behavior of the asphalts.

Penetration Ratio is one measure of temperature-susceptibility and according to Warden (1958), a minimum value of 25 % has been included in specifications by some organizations. Asphalt A and B have penetration ratios of 26 % and 36 % respectively; using the 25 % standard, both asphalts would meet the specification requirement but Asphalt A would be considerably more temperature susceptible than Asphalt B. This is not to be expected since Asphalt A is of a higher penetration grade than B.

The reasons why a temperature susceptibility clause has not been included in most specifications is, according to Mitchell (1961), because it has not been related to pavement performance. More recently, Shields (1964) has

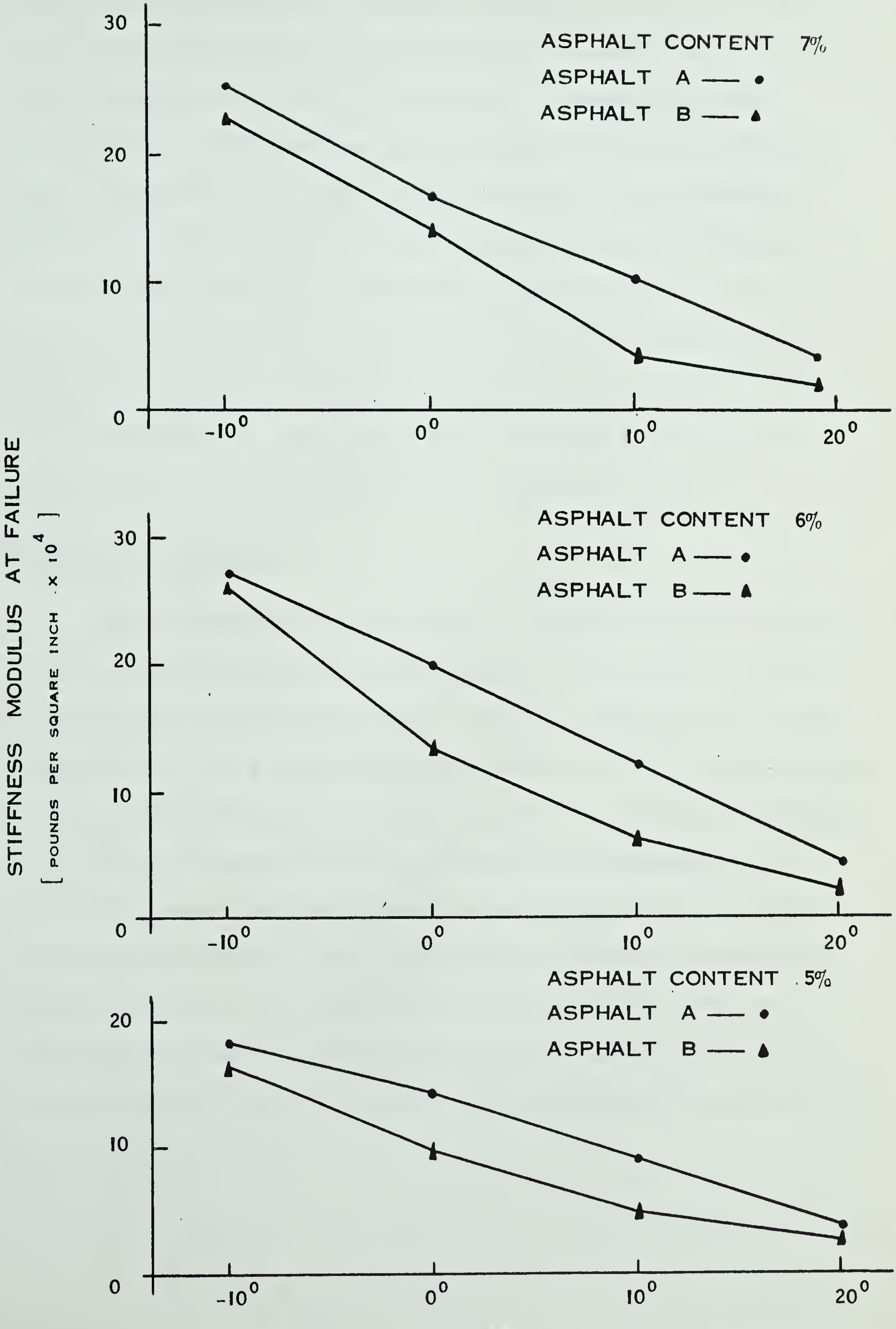



FIGURE 8 STIFFNESS MODULUS VERSUS TEMPERATURE

reported on transverse cracking of asphalt pavements and the detrimental effect of cracking on pavement performance. The frequency of cracking is highest in these sections using asphalts of high temperature-susceptibility although some exceptions have been noted. Asphalt A and Asphalt B have been used on many projects and have shown different cracking patterns, with pavements using Asphalt A consistently showing more cracks per mile than those using Asphalt B. Using the stress strain characteristics, the Stiffness Modulus, and temperature susceptibility, it is possible to differentiate between Asphalts A and B.

## Relation to Design

The difference in behavior of mixtures using asphalts from two sources have been compared on the basis of their relationships between strength, strain and stiffness versus temperature. Of a more practical interest is the application of this information to design of mixes. A possible approach is to impose certain limits on the three parameters of strength, strain and stiffness as measured or calculated in the testing program. By studying the effects of adjusting asphalt content upon strength, strain and stiffness over the working range it may be possible to exclude the use of certain asphalt cements at the lower temperature encountered during service life.

The idea of a limiting stress will not be considered. Monismith (1965) has suggested that induced stresses in asphalt concrete due to temperatures below 0° F may exceed the breaking strength of the material. For this reason, the idea of setting a limiting stress and adjusting asphalt content over the working range will not be considered.

Figure 9 shows the relationship between strain at failure and asphalt content. Even if information was available on the size of the limiting strain to be set and related to cracking in the field it would not be possible to differentiate between A and B at - 10° F because of their similar behavior at this temperature.

At higher temperatures it is possible to set a limiting strain which could be used to select or reject asphalts over the working range. Just what value is to be used is impossible to estimate because of the lack of information about failure strains that have occurred in the field. For example, at 0° F, if the limiting strain were set at 20, Asphalt A and B would be suitable because the limiting strain is never reached at this temperature. On the other hand, if the limiting strain were set at 25, Asphalt A could not be used between 5.3 and 6.7 percent, while Asphalt B could be used over the complete working range.

Figure 10 shows the relationship between stiffness and asphalt content. As discussed previously, information

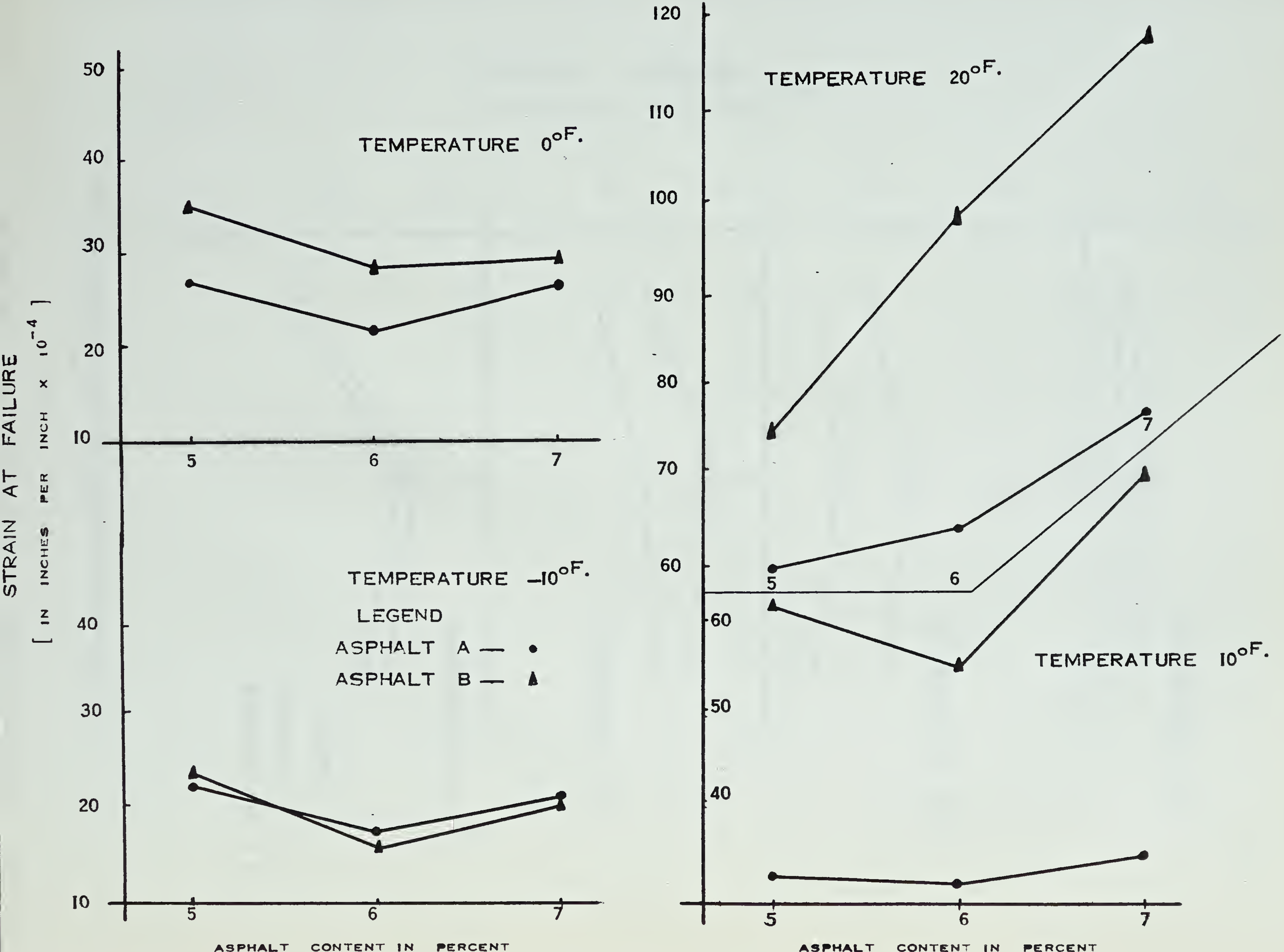



FIGURE 9 STRAIN VERSUS ASPHALT CONTENT

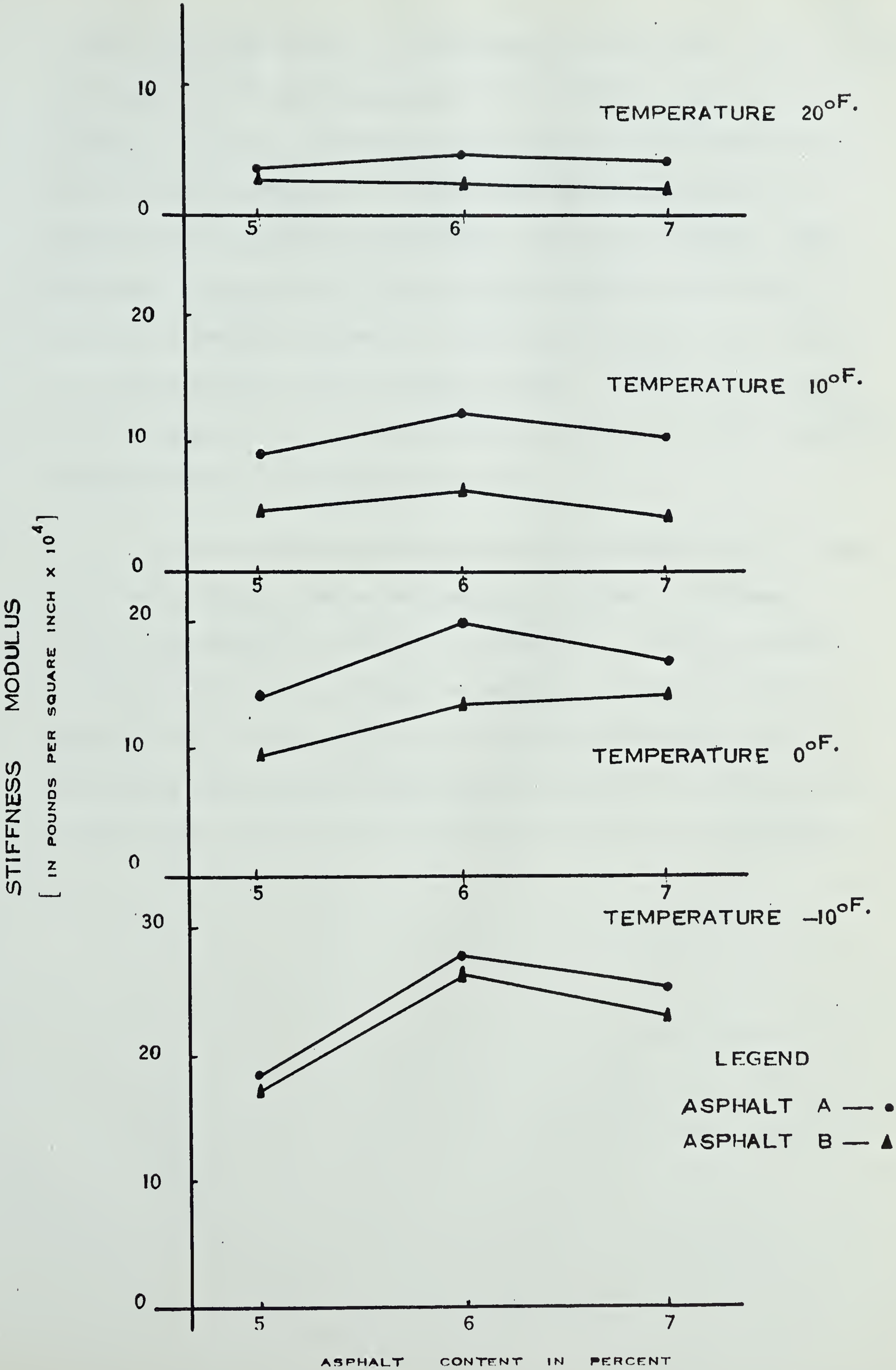



FIGURE 10 STIFFNESS MODULUS VERSUS ASPHALT CONTENT

on the limiting value of stiffness is not known. At - 10° F, the behavior of the asphalts is so similar that it is not possible to set a limiting stiffness at this temperature. At 0° F the difference in behavior of the asphalts is evident and a limiting stiffness could be assumed. For example, suppose the limiting stiffness is set at 15. Asphalt B never reaches the limiting stiffness and could be used over the whole working range. Asphalt A is too stiff and could not be used except for a very short working range between 5.0 and 5.2 percent.

The preceding paragraphs are presented only to show methods of using the relationships between stiffness, strain and asphalt content to establish limiting values that could be used in mixture designs. Whether such an approach is of any practical value is dependent upon obtaining information regarding the values of stiffness or strain that occurs in the field at the time of cracking.

# CHAPTER 7

## CONCLUSIONS AND RECOMMENDATIONS

The purpose of this study was to review the Marshall Mix Designs used in selected study areas and to develop the tensile splitting test as a means of measuring the difference in the low temperature characteristics of two asphalt types. The conclusions and recommendations arising from this investigation are presented in this chapter.

### Conclusions

1. No relationship was established between the frequency of cracking and the additional criteria of film thickness and voids bitumen index ratio; which were used as a further check on the Marshall Method of Mix Design.

2. The tensile splitting test is a valid test method for the rate of deformation used in the testing program at the temperatures of - 10°, 0° and 10° F. At 20° F, the shape of the stress versus strain curves indicated plastic behavior throughout the test and thus cannot be considered a valid test method at this temperature with this rate of deformation.

3. The differences in Asphalts A and B can be detected by observing:

   (a) the relation of stress and strain characteristics to temperature change; Asphalt A exhibits less strain and greater tensile strength than Asphalt B at all the test temperatures.

   (b) Stiffness Modulus varies with temperature. For Asphalt A, the stiffness modulus is greater than that of Asphalt B at all test temperatures.

   (c) Asphalt A is more susceptible to temperature change than Asphalt B.

4. Asphalts A and B have similar strain and stiffness values at - 10° F and no relation to design could be established. At higher test temperatures, 0° F and above, Asphalt A and B behave differently and it is possible to set a limiting strain or stiffness which would exclude the use of the harder asphalt at a certain asphalt content.

Recommendations

1. Procedures for running tensile splitting tests should be improved by closer control of specimen temperatures and by using devices for recording stress and strain throughout the test.

2. The stress strain characteristics of different mixes should be investigated. It is suggested that it be

done in conjunction with the Department of Highways and all future mix designs should include the provision of sufficient specimens for tensile testing.

3. The use of the Glass Transition Temperature ($T_G$) should be investigated for asphalts in current use to determine whether any relationship exists between the $T_G$ and temperatures at which low strain values occur in asphalt mixes.

4. Further investigations should determine the temperature at which cracking occurs in the field for different asphalt types. Correlating these results with tensile splitting tests on the same mix at the same temperature will provide the strain or stiffness modulus at failure. The practicability of establishing a limiting strain or stiffness modulus for different asphalt types can then be considered.

# LIST OF REFERENCES

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Asphalt Institute, "Mix Design Methods for Asphalt Concrete" Second Edition, January, 1962.

Barrell, E.M., Schmidt, R.J., Johnson, J.F., "Asphalt Transitions at Low Temperatures", Symposium on Asphalt Composition, Chemistry and Physics, American Chemical Society, April 1964.

Boyd, J.E., Folk, S.B., "Strength of Materials", Fifth Edition, McGraw-Hill Book Company, New York, 1950.

Campen, W.H., Smith, J.R., Erickson, L.G., Mertz, L.R., "The Relationships Between Voids, Surface Area, Film Thickness and Stability in Bituminous Paving Mixtures", Proceedings of the Association of Asphalt Paving Technologists, Volume 28, 1959.

Campen, W.H., Smith, J.R., Erickson, L.G., Mertz, L.R., "The Control of Voids in Aggregates for Bituminous Paving Mixtures", Proceedings of the Association of Paving Technologists, Volume 26, 1957.

Corbett, L.W., "Variation Studies in Density and Stability Measurements on Asphalt Aggregate Mixes", Proceedings of Association of Asphalt Paving Technologists, Volume 25, 1956.

Domaschuk, L., Skarsgard, P.S., Christianson, R.H., "Cracking of Asphalt Pavements Due to Thermal Contraction", Proceedings Canadian Good Roads Association, October, 1964.

Ferry, J.D., "Viscoelastic Properties of Polymers", John Wiley and Sons, Inc., 1961.

Frocht, M.M., "Photoelasticity", John Wiley and Sons, Inc., New York, 1948.

Frydman, S. "The Applicability of the Brazilian (Indirect Tension) Test to Soils", Australian Journal of Applied Science, Volume 15, 1964.

Goode, J.R., Lufsey, L.A., "Voids Permeability, and Film Thickness Related to Asphalt Hardening", Public Roads, October 1965.

Heukelom, W., Klomp, A.J.G., "Road Design and Dynamic Loading", Proceedings of Association of Asphalt Paving Technologists, February, 1964.

Hewitt, W.L., "Analyses of Flexible Paving Mixtures by Theoretical Design Procedure Based on Shear Strength", Highway Research Record Number 104, 1965.

Hode Keyser, J., Wade, P.F., "Variability in the Testing and Production of Bituminous Mixtures" Highway Research Record, Number 24, 1963.

Hveem, F.N., "Types and Causes of Failure in Highway Pavements", Highway Research Board, Bulletin 187, 1958.

Livneh, M., Shklarsky, E., "The Splitting Test for Determination of Bituminous Concrete Strength", Proceedings of the Association of Asphalt Paving Technologists, January, 1962.

Mitchell, D.W., "A Study of Alberta Asphalts", Unpublished Master of Science Thesis, University of Alberta, April 1961.

Mitchell, N.B., "The Indirect Tension Test for Concrete" Materials Research and Standards, Volume 1, 1961.

Mollard, D.G., "The Occurrence and Classification of Pavement Cracking and Ridging in Saskatchewan", Proceedings, Asphalt Seminar, University of Saskatchewan, Saskatoon, 1964.

Monismith, C.L., Secor, G.A., Secor, K.E., "Temperature Induced Stresses and Deformations in Asphaltic Concrete", Proceedings Association of Asphalt Paving Technologists, January 1965.

Monismith, C.L., "Asphalt Paving Mixtures, Properties, Design and Performance", Institute of Transportation and Traffic Engineering, University of California, October 1961.

Peltier, R., "Theoretical Investigation of the Brazilian Test", International Association of Testing and Research Laboratories for Materials and Structures No. 19, 1954.

Rader, L.F., "Investigations of the Physical Properties of Asphaltic Mixtures at Low Temperatures", Proceedings of the Association of Asphalt Paving Technologists, January 1935.

Sharan, V., "Tensile Properties of Asphalt Films", Unpublished Master's Thesis, University of Alberta Faculty of Graduate Studies, 1965.

Shields, B.P., Anderson, K.O., "Some Aspects of Transverse Cracking in Asphalt Pavements", Canadian Technical Asphalt Association, November 1964.

Shields, B.P., "Current Studies on Transverse Cracking of Asphalt Pavements", Proceedings of the Conference on Recent Developments in the Design and Construction of Asphalt Pavements, University of Alberta, February 1964.

Skarsgard, P.S., "The Effectiveness of Dummy Joints in Controlling Surface Cracking in Asphalt Pavements due to Thermal Contraction", Proceedings, Asphalt Seminar University of Saskatchewan, Saskatoon, 1964.

State of California, Division of Highways, Materials and Research Department, January 1942.

Stephens, J.E., Breen, J.J., "Split Cylinder Test Applied to Bituminous Mixtures", Materials Research and Standards, American Society for Testing Materials, (Abstract Only), Vol. 5, No. 5, 1965.

Thaulow, S., "Tensile Splitting Test and High Strength Concrete Test Cylinders", Journal of American Concrete Institute, January 1957.

Thompson, M.R., "Split-Tensile Strength of Lime-Stabilized Soils", Highway Research Record, Number 92, October 1965.

Tons, E., Krokosky, E.M., "Tensile Properties of Dense Graded Bituminous Concrete", Proceedings of the Association of Asphalt Paving Technologists, Volume 32, 1963.

Traxler, R.N., "Asphalt, Its Composition, Properties and Uses", Reinhold Publishing Corporation, New York, 1961.

Tuckerman, L.B., "Optical Strain Gauges and Extensometers", Proceedings of the American Society of Testing Materials - Twenty Sixty Annual Meeting, Vol. 23, Part II, 1923.

Tuckerman Optical Strain Gauge System, Instruction No. 750, American Instrument Co., Inc., 1958.

Vallerga, B.A., "On Asphalt Paving Performance", Proceedings of the Association of Asphalt Paving Technologists, Volume 24, 1955.

Van der Poel, C., "A General System Describing the Viscoelastic Properties of Bitumens and its Relation to Routine Test Data", Journal of Applied Chemistry, 4, May 1954.

Vokac, R., "Repeatability of Marshall Test by Analysis of Factorial Experiment Data", Proceedings of Association of Asphalt Paving Technologists, Volume 31, 1962.

Warden, W.B., "Quality of Asphalt Cements as Reflected by Specifications", Proceedings of the Canadian Technical Asphalt Association, November 1958.

Wright, P., "Comments on an Indirect Tensile Test on Concrete Cylinders", Magazine for Concrete Research Volume 7, October 1955.

Zube, E., Cechetini, J., "Expansion and Contraction of Asphalt Concrete Mixes", Highway Research Board, Number 104, December 1965.

# Appendix A

The Tensile Splitting Test

Determination of Gauge Length

The Tuckerman Strain Gauge

Preparation of Specimens For Testing

# THE TENSILE SPLITTING TEST

The tensile splitting test is conducted by placing a cylindrical specimen horizontally between two loading surfaces and loading the specimen along two opposite generatrices as shown in Figure A1. For elastic or brittle materials, weak in tension, the specimen fails in tension along the loaded diameter, A-B of the cylinder.

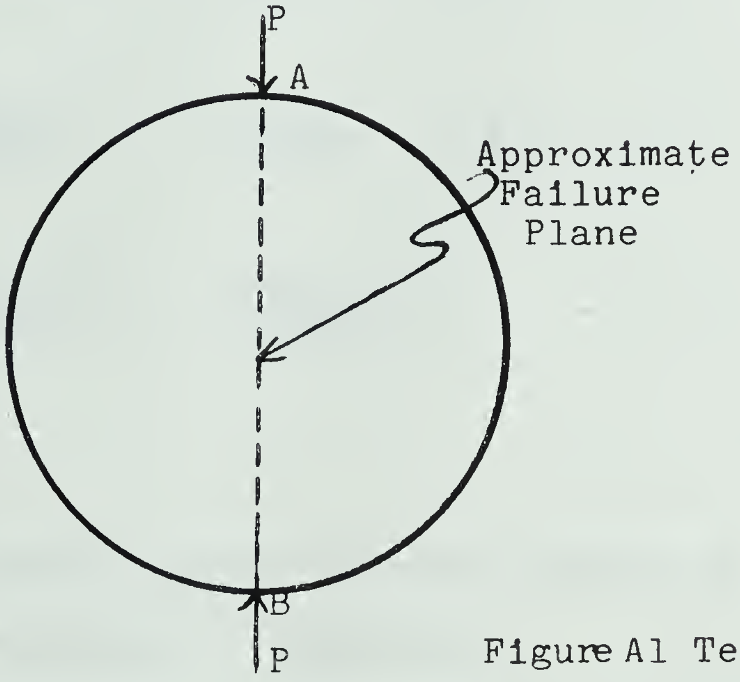


Figure A1 Tensile Splitting Test

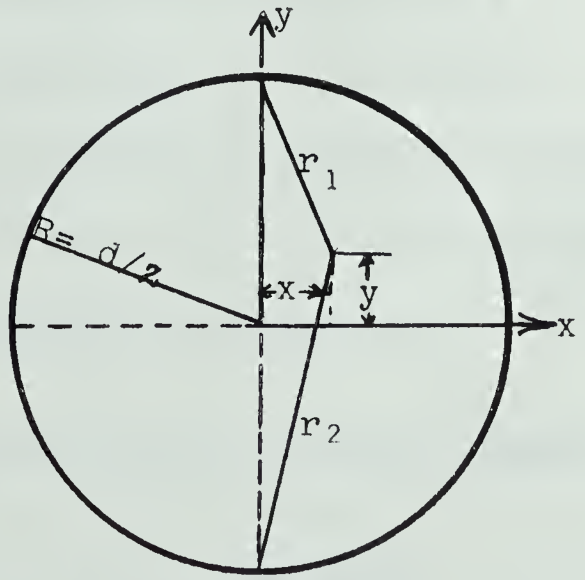


Figure A2 Coordinate System

The theoretical solution of the split test is based on the theory of elasticity. Assuming plane stress and considering a circular disk with concentrated loads on the diameter, Figure A2, three general equations can be used to express the stress conditions at all points within the disc. Frocht (1949). Frocht's equations are as follows:

$$\sigma_x = \frac{-2P}{\pi t} \left| \frac{(R-y)x^2}{r_1{}^4} + \frac{(R+y)x^2}{r_2{}^4} - \frac{1}{d} \right| \quad ------ (1)$$

$$\sigma_y = \frac{-2P}{\pi t} \left| \frac{(R-y)^3}{r_1{}^4} + \frac{(R+y)^3}{r_2{}^4} - \frac{1}{d} \right| \quad ------ (2)$$

$$\tau_{xy} = \frac{2P}{\pi t} \left| \frac{(R-y)^2 x}{r_1{}^4} - \frac{(R+y)^2 x}{r_2{}^4} \right| \quad ------ (3)$$

where

$\sigma_x$, $\sigma_y$, $\tau_{xy}$, = stress components with respect to rectangular coordinates

$x,y$ = rectangular coordinates

$P$ = load applied to specimen

$t$ = thickness of cylindrical specimen

$R$ = radius of cylindrical specimen

$r_1$ $r_2$ = location coordinates

Considering the horizontal diameter of the disc, the x-axis,

$$Y = 0,\ r_1 = r_2 = \sqrt{x^2 + r^2}$$

and the stress equations simplify to:

$$\sigma_x = \frac{2P}{\pi dt}\left[\frac{d^2 - 4x^2}{d^2 + 4x^2}\right]^2 \text{ - - - - - - - - - - - - (4)}$$

$$\sigma_y = \frac{-2P}{\pi dt}\left[\frac{4d^4}{(d^2 + 4x^2)^2} - 1\right] \text{ - - - - - - - - -(5)}$$

$$\tau_{xy} = 0 \text{ - - - - - - - - - - - - - - - - - - - - (6)}$$

The vertical stress, $\sigma_y$ , along the X axis is always a compressive stress and varies from a maximum at the center to zero at the circumference. At the center, the magnitude of $\sigma_y$ is - $6P/\pi d$ and the accompanying horizontal stress, $\sigma_x$, is a tensile stress equal to $2P/\pi d$. This indicates that the material tested must have a compressive strength at least three times its tensile strength in order to ensure a tensile failure. The stress distribution along the X axis is shown in Figure A3.

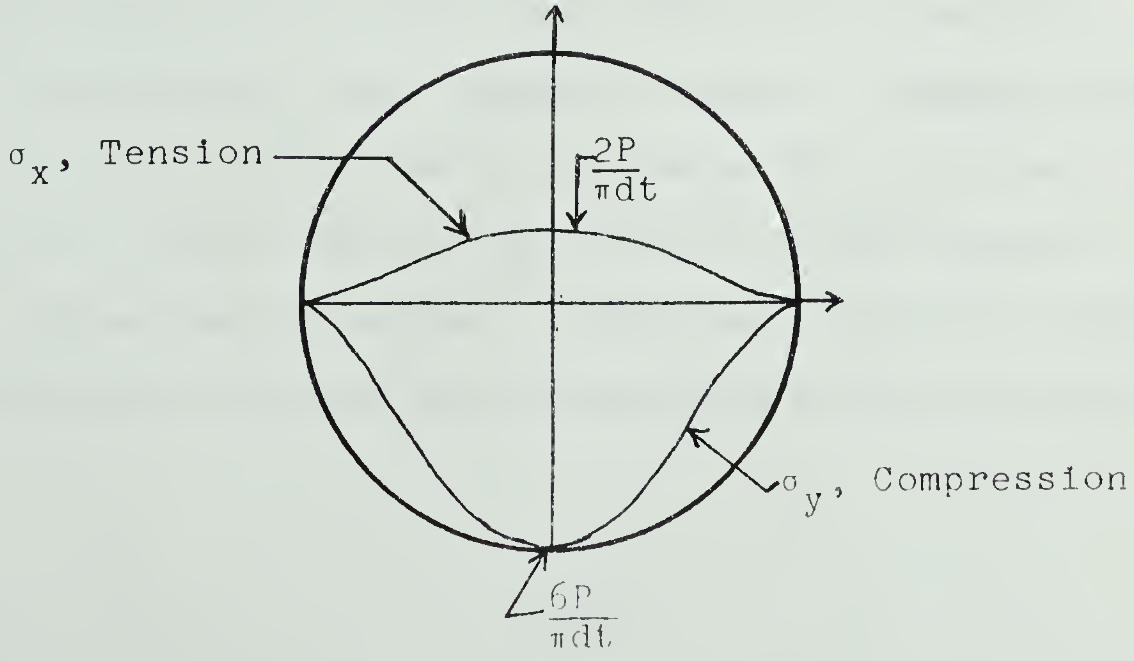


FIGURE A3 STRESS DISTRIBUTION ON X-AXIS

For a vertical plane through the center of the disk along the Y-axis, the equations reduce to:

$$\sigma_x = \frac{2P}{\pi dt} \text{ - - - - - - - - - - - - - - - - - - - - (7)}$$

$$\sigma_y = \frac{-2P}{\pi t} \left[ \frac{2}{d-2y} + \frac{2}{d+2y} - \frac{1}{d} \right] \text{ - - - - - - - - - (8)}$$

$$\tau_{xy} = 0 \text{ - - - - - - - - - - - - - - - - - - - - - - - - (9)}$$

The horizontal tensile stresses, $\sigma_x$ along the vertical plane have a constant value of $2P/\pi dt$ and the vertical compressive forces vary from $-6P/\pi dt$ at the center to infinity at the loading points. With a concentrated load, the specimen will fail at the load points due to the compressive stresses and not in the central part of the specimen due to the tensile stress. It has been shown with photoelastic studies that this point of maximum stress can be moved away from the boundary by using a distributed load applied through a loading strip. Also, the distributed load will change the $\sigma_x$ stress from tension to compression in the vicinity of the loading plate. Experimental evidence indicates that this condition is sufficient to retard the compressive failure at the

load points and the cylinder fails due to the tensile stresses at the center.

A schematic representation of the test specimen and the loading strips is shown in Figure A4. The use of loading strips alters the stress distribution in the specimen.

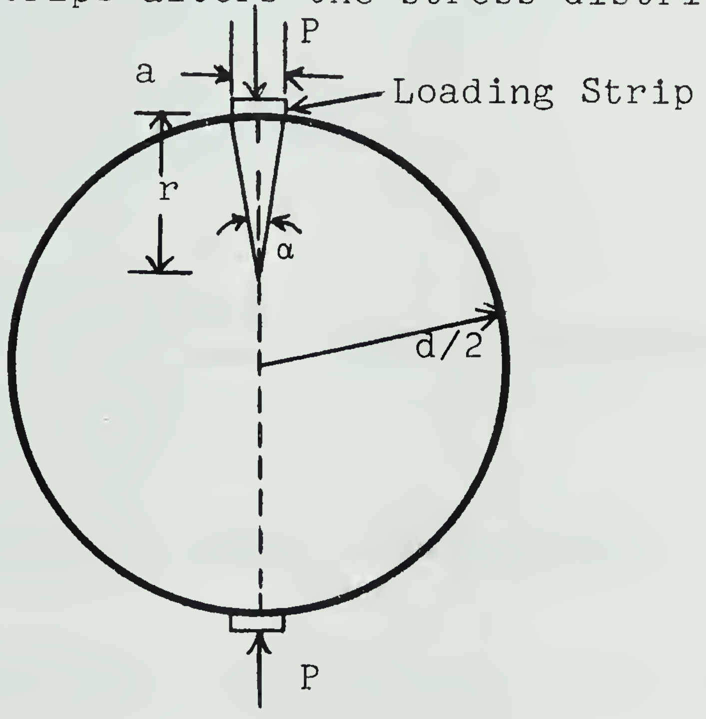


FIGURE A4 LOADING STRIPS

According to Wright (1955), the horizontal stress distribution $\sigma_x$ along the vertical diameter is closely approximated by:

$$\sigma_x = \frac{-2P}{\pi t d} \left[ 1 - \frac{d}{2a} (\alpha - \sin\alpha) \right] \qquad (10)$$

providing the loading strip, a, is less than d/10. The resulting horizontal stress distribution on the vertical diameter is shown in Figure A5.

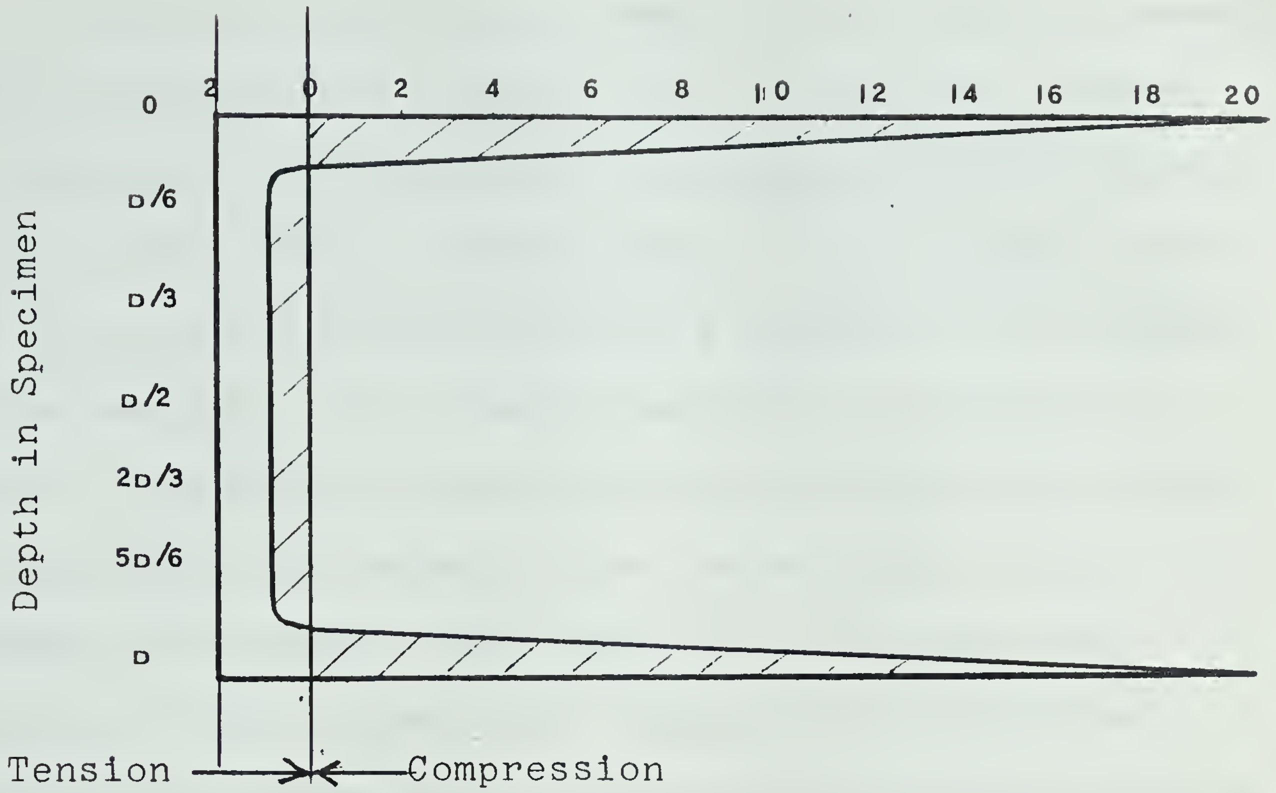


FIGURE A5 HORIZONTAL STRESS DISTRIBUTION ON Y-AXIS

Additional work by Peltier (1954), has shown that the tensile stresses remain uniform over a reasonable portion of the diameter providing the width of the loading strips is less than d/5. It thus appears that when loading strips of the width suggested are utilized, the tensile stresses are approximately equal to 2P/ dt over a major part of the loaded diameter Wright (1955).

Determination of Gauge Length

The Tuckerman Strain Gauge can be used for measuring strain between gauge lengths from 1/4 inch to 2 inches. As illustrated in Figure A3 of this Appendix, the tensile stress across the horizontal axis of the specimen varies from zero at the outside edge to a maximum at the center of the specimen. For elastic materials, according to Hooke's Law, stress is proportional to strain and since the specimens are tested at low temperatures it is assumed that elastic conditions exist for each test. Therefore it is assumed that strain also varies from zero at the outside edge to a maximum value at the center of the specimen.

It is impractical to use the smallest gauge length (1/4 inch) to measure maximum strain because of the location of the resulting fracture. This usually occurs in a band up to 1/2 inch on either side of the vertical axis. The tensile stress at a point 1/2 inch from the vertical axis can be calculated as being 78 % of the maximum stress in the specimen. Strain measurements based on this center inch would result from a tensile stress that is approximately 90 % of the maximum tensile stress along the vertical axis assuming a linear relationship in stress distribution between these points. If necessary, a correction factor could be applied to the strain readings

to obtain the maximum strain resulting from the maximum tensile stress at the center of the specimen.

For the wider gauge widths, up to 2 inches, the amount of strain measured would be too great for the range of strain the Tuckerman Strain Gauge is capable of measuring.

Therefore, the 1 inch gauge length was used throughout the testing program.

# The Tuckerman Strain Gauge

## Introduction

The Tuckerman Strain Gauge is an optical strain gauge with high sensitivity and capable of measuring accurately strains as small as 2 micro inches per inch. A complete description of the instrument has been given by Tuckerman 1923 and for this reason only a brief description of the instruments is given here. The main components of the instrument are the autocollimator and the strain gauge. A diagrammatic sketch is shown in Figure A6.

## The Autocollimator

The autocollimator consists of a highly corrected, two component objective lens having a focal length of about 250 mm., a reticule having a uniformly graduated scale, a fiduciary line and vernier accurately positioned to be in the focal plane of the objective lens, a lamp for illuminating the fiduciary line and vernier, and a Ramsden Type eyepiece. The collimator is designed such that the reticule and objective lens can be removed for cleaning and replaced without changing the calibration of the instrument.

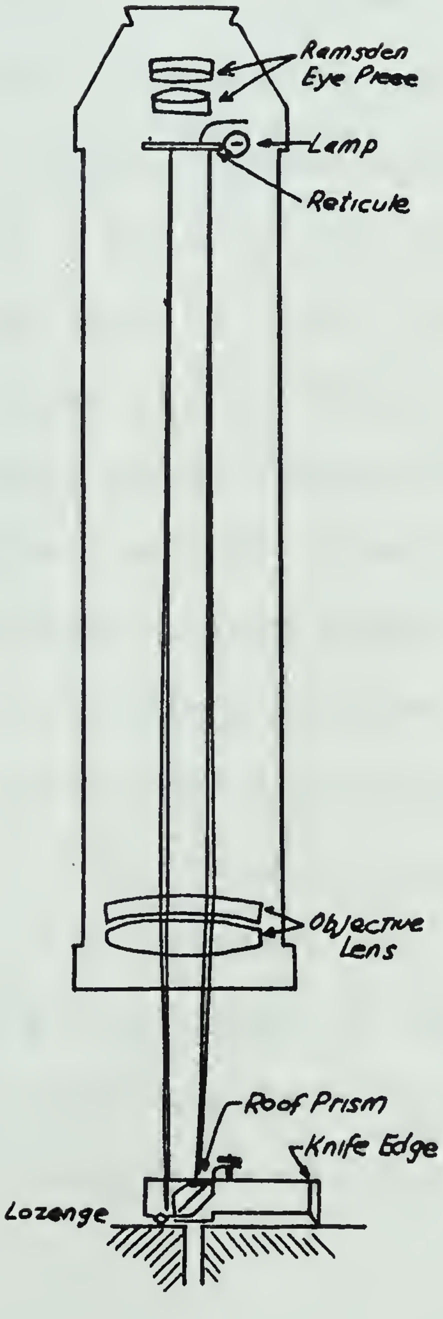


FIGURE A6 - DIAGRAMMATIC SKETCH OF TUCKERMAN AUTOCOLLIMATOR AND STRAIN GAUGE

The Strain Gauge

The strain gauge consists of a steel body to which is attached a knife edge and parts of the mirror system, which consists of the lozenge and the roof prism. The movable mirror is one surface of the lozenge and rotates about an axis perpendicular to the axis of the gauge. Two fixed, mutually perpendicular mirrors are the roofs of a prism which can be rotated by means of an adjusting knob to reset the gauge. The prism rotates about an axis perpendicular to the plane which bisects the dehedral angle formed by the roof surfaces. As the specimen deforms, the lozenge rotates about an axis perpendicular to this plane. The roof prism contains a third reflecting surface, known as the flash surface, which is parallel to the line of intersection of the two roof surfaces and perpendicular to the plane which bisects the dehedral angle formed by the roof surfaces. When the instrument is read, it is necessary to have the image reflected from the flash surface in the field of view, to avoid the introduction of an error.

How the Instrument Works

Light from the fiduciary line and vernier scale is rendered parallel in passing through the objective lens

of the autocollimator and falls upon both the lozenge and the roof prism. Only the light which falls upon the roof prism is shown in Figure A6. This light is reflected by the two roof surfaces of the prism and by the surface of the lozenge, back into the objective lens to form an image on the scale of the reticule. Deformation of the surface to which the gauge is attached results in a rotation of the lozenge and will move the image along the scale.

The reflected image shown in Figure A6 is the compression image. The tension image, not shown in the figure is formed by the light which first falls on the lozenge and is not visible in the eyepiece because it lies as far to the right of the fiduciary line as the compression image shown lies to the left.

As the lozenge is rotated to move the compression image towards the fiduciary line both the visible compression image and the invisible tension image will approach the fiduciary line. As the rotation continues, the compression image will disappear from view after passing the 0 scale line and the tension image will appear and move up the scale. Since the distance from the fiduciary line to the zero scale line is equivalent to 5 divisions neither image is visible for a rotation of the lozenge corresponding to 10 divisions.

## The Gauge Range

The redicule scale consists of 125 graduations, twenty-five of which are numbered. Each numbered division represents 0.001 radian rotation of the lozenge. This angle is calculated from the equation

$$\frac{d}{f} = \tan 2\alpha = 2\alpha \text{ (for small angles measured) in radians} \quad (1)$$

where d is in millimeters between number divisions, f is the focal length of the objective lens in millimeters, and $\alpha$ is the rotation of the lozenge in radians. Since the distance between the numbered divisions is 0.5 mm. and the focal length is 250 mm. one numbered division represents an angular rotation of

$$\frac{0.5}{250 \times 2} = 0.001 \text{ radian.}$$

Therefore, the entire scale of 25 numbered divisions corresponds to 0.025 radian rotation of the lozenge.

The relation between the 25 numbered divisions and strain can be calculated from the equation

$$\text{Strain} = \frac{LR}{1000\ E} \text{ - - - - - - - - - - } \quad (2)$$

where L represents lozenge size, R represents total numbered divisions, and E the gauge length. To calculate actual strains based on readings, an autocollimator calibration

factor and a lozenge calibration factor (both factory supplied) must be used because the actual focal length of objective lens and lozenge diagonal distance vary within small limits from the true or marked values. Thus, to obtain strain in inches per inch equation (2) is reversed to

$$\text{Strain (inches per inch)} - \frac{LRFA}{1000E} \text{ - - - - - - - } (3)$$

where L, R, and E are as before, F and A are the calibration factors for lozenge and autocollimator respectively.

The range of strain can be extended to cover 102 Numbered divisions by maintaining a constant load and twisting the adjusting knob of the roof prism. (Not possible with asphalt materials because of the slight plastic deformation which results under constant load even in the low temperature range.) Table A 1 shows the complete range of strain that can be attained with the Tuckerman Strain Gauge using different gauge lengths and the 0.2 inch and 0.4 inch lozenges.

TABLE A I

Standard Tuckerman Gauge (a)

| Gauge Length (Inches) | Lozenge Size: 0.2" across diagonal | | | | Lozenge Size: 0.4" across diagonal | | | |
|---|---|---|---|---|---|---|---|---|
| | Range of Deformation (inches) | | Range of Strain (inches/inch) | | Range of Deformation (inches) | | Range of Strain (inches/inch) | |
| | 23 Div. | 102 Div. | 23 Div. | 102 Div. | 23 Div. | 102 Div. | 23 Div. | 102 Div. |
| 1/4 | 0.0046 | 0.0204 | 0.0184 | 0.0816 | ---- | ----- | ----- | ----- |
| 1/2 | 0.0046 | 0.0204 | 0.0092 | 0.0408 | 0.0092 | 0.0408 | 0.0184 | 0.0816 |
| 1 | 0.0046 | 0.0204 | 0.0046 | 0.0204 | 0.0092 | 0.0408 | 0.0092 | 0.0408 |
| 2 | 0.0046 | 0.0204 | 0.0023 | 0.0102 | 0.0092 | 0.0408 | 0.0046 | 0.0204 |
| 3 | 0.0046 | 0.0204 | 0.00153 | 0.0068 | 0.0092 | 0.0408 | 0.00307 | 0.0136 |
| 4 | 0.0046 | 0.0204 | 0.00115 | 0.0051 | 0.0092 | 0.0408 | 0.0023 | 0.0102 |

(a) Data obtained from Tuckerman Strain Gauge Instruction No. 750, American Instrument Co., Inc., 1958.

PREPARATION OF SPECIMENS FOR TESTING

(1) Inspect specimen closely. Many of the specimens contained large voids along the curved surfaces. Since specimens are to be subjected to a compressive force, specimens were oriented such that the flaws were outside the areas in contact with the loading blocks. Specimens were then marked along the horizontal and vertical diameters for correct positioning in the testing machine.

(2) Setting of Gauge Marks. Aluminum blocks, 1/4" x 1/4" x 1/8" with a scribed line were fixed to the specimen with a drop of asphalt cement. Asphalt cement, rather than other adhesives were used because of the low temperature of the testing. Gauge marks were 1 inch apart along the horizontal axis on either side of the vertical axis. A tool with a gauge length of 1 inch between knife edge was placed in the scribed gauge marks to ensure a proper gauge length.

(3) Cooling of specimens. All specimens were placed in the frost room and cooled to the test temperatures for a period of 24 hours. Temperature of the frost room was maintained constant within 3 or 4 fahrenheit degrees of the testing temperature.

(4) Attaching Strain Gauge to Specimen. Prior to removal of the specimen from the frost room the strain gauge is positioned on the specimen and held in place with elastic bands. The knife edges of the strain gauge are firmly seated in the scribed lines of the gauge marks.

(5) The specimen and strain gauge are then placed in the testing machine as shown in Figure A 7. Blocks of plywood 3" x 1/2" x 1/4" are placed on the top and bottom of the sample along the vertical axis. Loading head of the testing machine is lowered such that the specimen is held firmly in position. Loading dial is adjusted to zero load and a reading taken on the dial gauge that records the vertical movement of the loading head during the test. A timer is also set at zero. The loading needle valve is set at 10 divisions which corresponds to a strain rate in the loading head of .045 inches per minute.

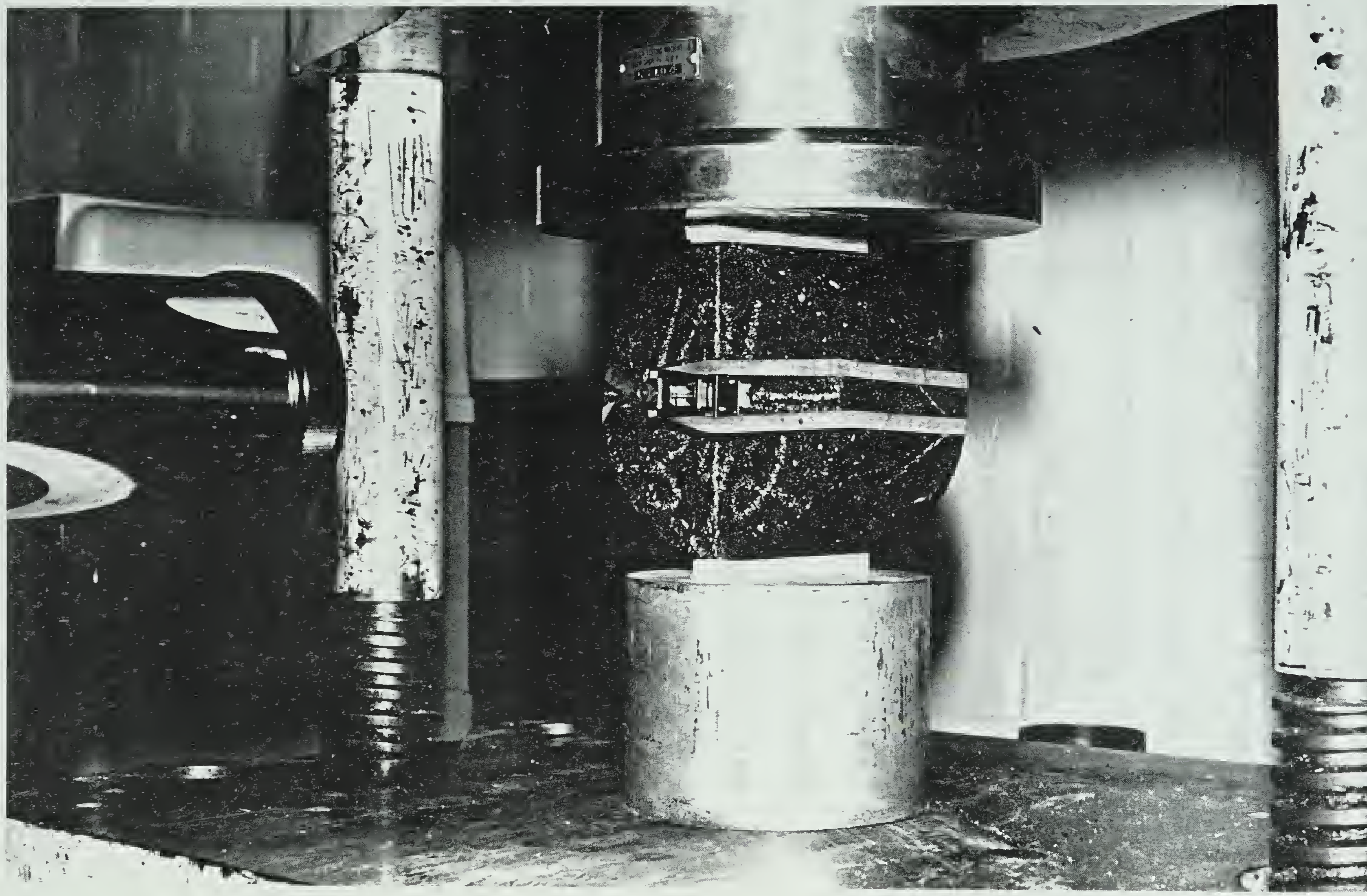

FIGURE A7 STRAIN GAUGE IN POSITION FOR TESTING

(6) Adjustment of the Strain Gauge. Just prior to the testing, the position of the reflected image is observed in the eyepiece of the collimator. If the reflected image is in view, the lozenge tip is held in place while the knife edge is moved in the direction that simulates a tensile stress. If the reflected image moves up scale, the roof prism is adjusted until movement of the knife edge results in a downscale image movement. If no reflected image is observed in the eyepiece, the roof prism must be adjusted until the image appears. Just prior to the test the roof prism is again adjusted to position the reflected image opposite the twenty-five division mark. This procedure ensures that the strain gauge is adjusted to record the maximum range of strain.

(The above operations take about two minutes after the specimen has been removed from the frost room; if more time is required to adjust the strain gauge, the specimen is returned to the frost room, another is selected and the operation completed.)

(7) Running the Test. A reading of the reflected image is taken at zero load and the test is started. Tuckerman strain gauge readings are recorded for every 200 kilogram increment except near the end

of the test when the increment is reduced to 100 kilograms between readings. If possible the strain at failure at the time of maximum load is also recorded. Throughout the test, the time movement of loading head, and applied load dial readings are recorded. This latter data is not necessary but serves a check to ensure the rate of applied strain and applied load are approximately the same for each test.

Appendix B

Sample Calculations

# SAMPLE CALCULATIONS

1. The Section selected to illustrate the calculations for the Marshall Mix Design study is taken from the first line of Table VI for Highway 2-G4 (Design Only)

(a) Surface Area and Film Thickness

Specific Gravity Effective of Aggregate = 2.625

Specific Gravity of Asphalt = 1.025

Asphalt Absorption = 0.23 %

Gradation of Aggregate

| Sieve No. | Percent Passing | x | Surface Area Factor Square Feet/100 lbs. | = | Surface Area |
|---|---|---|---|---|---|
| 3/4" | 100 | | 2 | | 200 |
| #4 | 51 | | 2 | | 102 |
| #8 | 42 | | 4 | | 168 |
| #16 | 34 | | 8 | | 272 |
| #30 | 26 | | 14 | | 364 |
| #50 | 19 | | 30 | | 570 |
| #100 | 12 | | 60 | | 720 |
| #200 | 64 | | 100 | | 640 |

Total 3036 sq. ft. 100 lbs

= 30.36 sq. ft. per 100 lbs of aggregate

Effective Surface Area - $30.36 \times \frac{2.65}{2.625} = 30.60$ sq. ft.

Effective Asphalt Content = 5.70 - 2.3 = 5.67 %

$$\text{Film Thickness} = \frac{\text{Pounds of Asphalt Per Pound of Aggregate}}{\text{Surface Area per Pound of Aggregate}}$$

$$\text{(in microns)} = \frac{.0547 \times 453 \times 10^{4}}{30.60 \times 1.025 \times 930} = 8.5 \text{ microns}$$

(b) $\text{Bitumen Index} = \frac{\text{Effective Asphalt Content in Percent}}{\text{Surface Area per pound of Aggregate}}$

$$= \frac{5.47}{30.60} = 1.8 \times 10^{-3}$$

(c) $\text{Voids - Bitumen Index Ratio} = \frac{\text{Percent Air Voids}}{\text{Bitumen Index} \times 10^{3}}$

$$= \frac{3.4}{1.8 \times 10^{-3}} \times 10^{3} = 1.89$$

2. Calculations with reference to the Tensile Splitting Test.

Test Condition 6 % Asphalt A - 10°F will be used to show the calculations involved. Laboratory Data Sheets for these tests are contained in Appendix C.

(a) $\text{Tensile Splitting Stress} = \frac{2P}{\pi d t}$

where P = total load in pounds

d = diameter in inches

t = thickness in inches

| Test No. | | Load In Kilograms | Load In Pounds | Diameter In Inches | Thickness In Inches | Tensile Stress Pounds Per Sq. Inch |
|---|---|---|---|---|---|---|
| 13 | x | 2700 | 5960 | 4 | 2.515 | 378 |
| 14 | | 3300 | 7280 | 4 | 2.559 | 453 |
| 15 | | 3540 | 7820 | 4 | 2.584 | 498 |
| 83 | | 3200 | 7060 | 4 | 2.563 | 440 |
| 84 | | 3600 | 7940 | 4 | 2.537 | 496 |
| 109 | | 3820 | 8430 | 4 | 2.539 | 525 |

(All Tests Used Except Nos. 13 and 83) Total = 1972

Mean Tensile Stress at Failure = $\frac{1972}{4}$ = 493 psi

(b) <u>Strain at Failure</u>

Strain (inches per inch) = $\frac{FALR}{1000E}$

F (lozenge calibration factor) = 1.003

A (collimator calibration factor) = 1.003

L (lozenge size in inches) = 0.2

R (traversed divisions)

E (gauge length) = 1.0

The only variable in this formula is R.

Therefore $\frac{FAL}{1000E} = \frac{1.003 \times 1.003 \times 0.2}{1000 \times 1} = 2.012 \times 10^{-4}$

inches/in per division

| Test No. | | R | $\frac{FAL}{1000E}$ | Strain at Failure $\times 10^{-4}$ |
|---|---|---|---|---|
| 13 | X | 22.6 | $2.012 \times 10^{-4}$ | 45.2 |
| 14 | | 11.0 | " | 22.0 |
| 15 | | 8.0 | " | 16.0 |
| 83 | X | 3.5 | " | 7.0 |
| 84 | | 6.0 | " | 12.0 |
| 109 | | 10.0 | " | 22.0 |

All Tests Used Except Nos. 13 and 83   Total = 72

Mean Strain at Failure = $\frac{72}{4} = 18 \times 10^{-4}$ in./in.

(c) <u>Stiffness Modulus</u>

$$\underline{\text{Stiffness Modulus}} = \frac{\text{Tensile Stress}}{\text{Strain}}$$

$$= \frac{493}{18 \times 10^{-4}} = 27.2 \times 10^{-4} \text{ psi}$$

Appendix C

Marshall Mixture Design Data

Data Sheets For Test Condition 6 % Asphalt A at -10°F

| DEPARTMENT OF HIGHWAYS TESTING LABORATORY | Project | P.R.S.P. No. |
|---|---|---|
| | Oil Co. | P.R.S.P. Name |
| | M.S.T. No. 392 | (From) Pit Name PONOKA NO. 3 |
| Summary Sheet | Engineer | Pit No. 3077 – 210 |
| MARSHALL STABILITY TEST | Location | Date Reported |

RESULTS FROM TESTS

| | | | | | | |
|---|---|---|---|---|---|---|
| Asphalt Content % | 4.7 | 5.3 | 5.9 | 6.5 | 7.2 | |
| Density Lbs./Cu. Ft. | 139.3 | 140.2 | 140.9 | 141.5 | 142.1 | |
| Stability Lbs. | 1030 | 1100 | 1125 | 1230 | 1440 | |
| Flow 0.01 Inches | 9.7 | 9.8 | 8.7 | 9.4 | 12.4 | |
| % Air Voids Total Mix | 8.7 | 7.5 | 6.2 | 5.1 | 3.9 | |
| % Voids Mineral Aggregate | 16.8 | 16.8 | 16.8 | 16.9 | 17.1 | |
| % Voids Filled with Asphalt | 48.2 | 55.4 | 63.1 | 69.8 | 77.2 | |

RESULTS FROM CURVES DRAWN

| | | | | | | |
|---|---|---|---|---|---|---|
| Asphalt Content % | 4.7 | 5.3 | 5.9 | 6.5 | 7.2 | |
| Density Lbs./Cu. Ft. | 139.3 | 140.2 | 140.9 | 141.5 | 142.1 | |
| Stability Lbs. | 1030 | 1075 | 1150 | 1260 | 1440 | |
| Flow 0.01 Inches | 9.7 | 8.8 | 8.7 | 9.4 | 12.4 | |
| % Air Voids Total Mix | 8.7 | 7.5 | 6.2 | 5.1 | 3.9 | |
| % Voids Mineral Aggregate | 16.8 | 16.8 | 16.8 | 16.9 | 17.1 | |
| % Voids Filled With Asphalt | 48.2 | 55.4 | 63.1 | 69.8 | 77.2 | |

| Passing | % Washed | | |
|---|---|---|---|
| 3/4" | | | 100 |
| No. 4 | | | 51 |
| No. 10 | | | 36 |
| No. 40 | | | 15 |
| No. 200 | | | 7.4 |
| | | | |

| | |
|---|---|
| Suggested Asphalt % | 6.7 |
| Density Lbs./Cu. Ft. | 141.7 |
| Stability Lbs. | 1310 |
| Flow 0.01 Inches | 9.5 |
| % Voids Mineral Aggregate | 17.0 |
| % Voids Filled With Asphalt | 72 |
| % Air Voids Total Mix | 4.8 |
| Asphalt Absorption % | 0.57 |
| Apparent Specific Gravity of Aggregate | 2.684 |
| Bulk Specific Gravity of Aggregate | 2.563 |
| Effective Specific Gravity of Aggregate | 2.600 |
| Specific Gravity of Asphalt Used in Test | 1.024 |

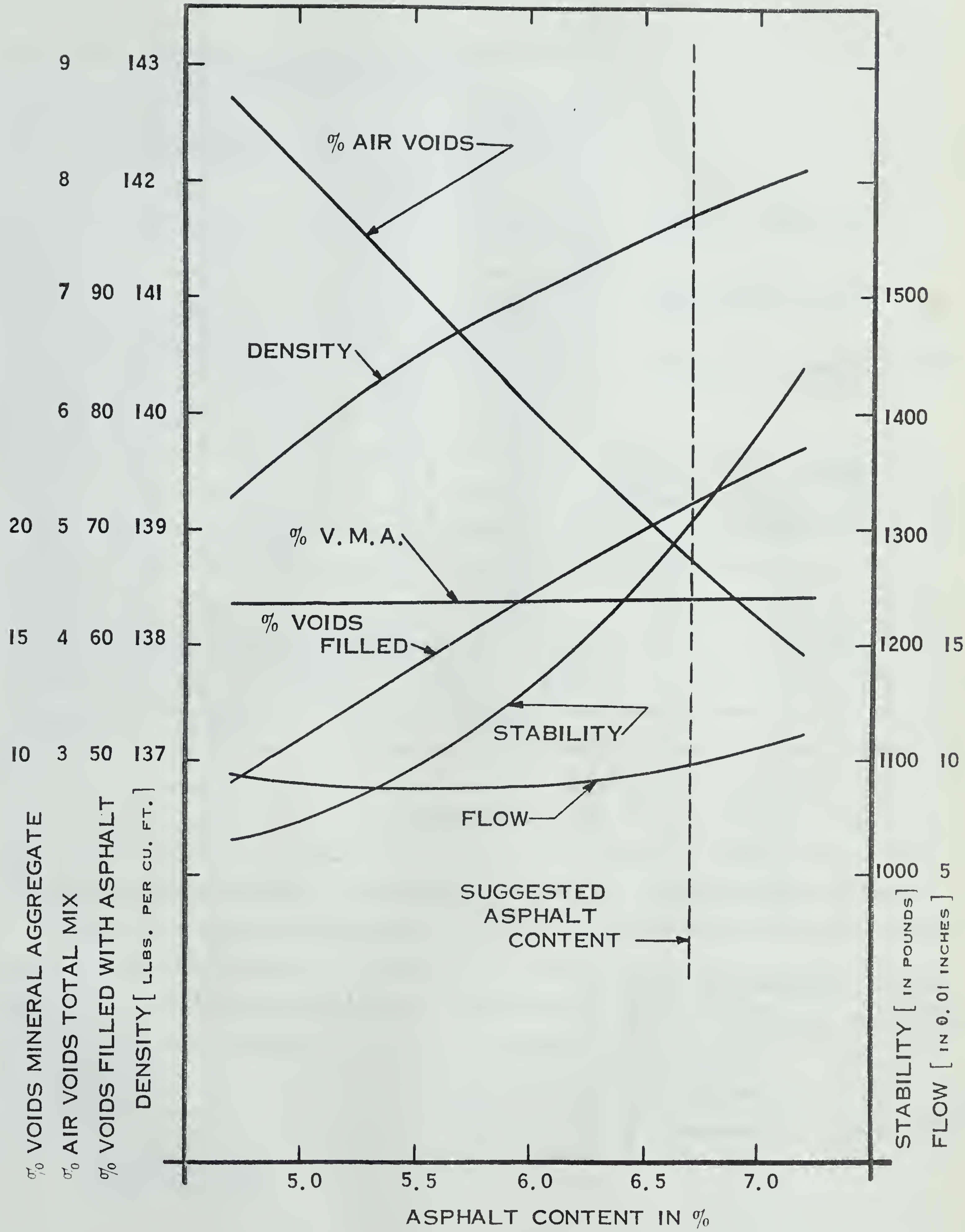


FIGURE C2 ASPHALT CONTENT VERSUS STABILITY, % AIR VOIDS, % VOIDS FILLED, DENSITY AND FLOW

| Load in Kgs. | STRAIN ACROSS SAMPLE | | | Tension ( psi ) |
|---|---|---|---|---|
| | Fiduciary Reading | Divisions Traversed | Strain in. per in. $\times 10^{-4}$ | |
| 0 | 12.6 | 0 | 0 | 0 |
| 200 | | | | |
| 400 | | | | |
| 600 | 12.2 12.1 | 0.4 0.5 | 0.8 1.0 | 70 84 |
| 800 | 11.7 | 0.9 | 1.8 | 112 |
| 1000 | 11.1 | 1.5 | 3.0 | 140 |
| 1200 | 10.0 | 2.6 | 5.2 | 168 |
| 1400 | 9.2 | 3.4 | 6.8 | 196 |
| 1600 | 8.2 | 4.4 | 8.8 | 224 |
| 1800 | 7.0 | 5.6 | 11.2 | 252 |
| 2000 | 5.4 | 7.2 | 14.4 | 281 |
| 2200 | 3.4 | 9.2 | 18.4 | 308 |
| 2400 | 2.0/0.0 | 10.6/12.6 | 21.2/25.2 | 337 |
| 2600 | 0.0/4.0 | 16.6 | 33.2 | 364 |
| 2800 | 0.0 | 22.6 | 45.2 | 378 |
| | | | | |
| | | | | |
| | | | | |
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| | | | | |
| | | | | |
| | | | | |
| | | | | |

Fail at 2700

Date 22 Jan 66

Specimen No. 13

Temp. of sample $-10^{\circ}F$.

| Time (min) | Dial Reading | Strain per min | Load |
|---|---|---|---|
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |

Time to Failure 6 min.

Rate of Strain 0.04"/min

Thickness 2.515"

$\gamma$ measured 144.1#/cu.ft

$\gamma$ immersed 144.7#/cu.ft.

| Load in Kgs. | STRAIN ACROSS SAMPLE Fiduciary Reading | Divisions Traversed | Strain in. per in. $\times 10^{-4}$ | Tension psi. |
|---|---|---|---|---|
| 0 | 24.0 | 0 | 0 | 0 |
| 200 | 23.9 | 0.1 | 0.2 | 27 |
| 400 | 23.9 | 0.1 | 0.1 | 55 |
| 600 | 23.7 | 0.3 | 0.6 | 83 |
| 800 | 23.4 | 0.6 | 1.2 | 110 |
| 1000 | 23.0 | 1.0 | 2.0 | 137 |
| 1200 | 22.6 | 1.4 | 2.8 | 165 |
| 1400 | 22.2 | 1.8 | 3.6 | 193 |
| 1600 | 21.9 | 2.1 | 4.2 | 220 |
| 1800 | 21.6 | 2.4 | 4.8 | 248 |
| 2000 | 21.2 | 2.8 | 5.6 | 276 |
| 2200 | 20.8 | 3.2 | 6.4 | 302 |
| 2400 | 20.4 | 3.6 | 7.2 | 330 |
| 2600 | 20.0 | 4.0 | 8.0 | 357 |
| 2800 | 19.4 | 4.6 | 9.2 | 385 |
| 3000 | 18.6 | 5.4 | 10.8 | 413 |
| 3200 | 16.5 | 7.5 | 15.0 | 440 |
| 3400 | 13.0 | 11.0 | 22.0 | 453 |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |

Date 22 Jan 66

Specimen No.14

Temp. of Sample - $10^{\circ F}$.

| Time (min) | Dial Reading | strain per min | Load |
|---|---|---|---|
| 0 | 885 | | |
| 1 | 853 | .032 | |
| 2 | 810 | .043 | |
| 3 | 765 | .045 | |
| 4 | 720 | .045 | |
| 5 | | | |
| 6 | 595 | | |
| | | | |
| | | | |
| | | | |
| | | | |

Time to Failure 6 min

Rate of Strain 0.04"/min.

Thickness 2.559 "

$\gamma$ measured 141.0#/cu.ft.

$\gamma$ immersed 141.8#/cu.ft.

Sample cracked both on both sides

Failure @ 3300 --13.0

| Load in Kgs. | STRAIN ACROSS SAMPLE Fiduciary Reading | Divisions Traversed | Strain in. per in. x $10^{-4}$ | Tension psi. |
|---|---|---|---|---|
| 0 | 19.0 | 0 | 0 | 0 |
| 200 | | | | |
| 400 | 18.5 | 0.5 | 1.0 | 55 |
| 600 | 18.4 | 0.6 | 1.2 | 83 |
| 800 | 18.2 | 0.8 | 1.6 | 110 |
| 1000 | 17.6 | 1.4 | 2.8 | 138 |
| 1200 | 17.2 | 1.8 | 3.6 | 165 |
| 1400 | 16.8 | 2.2 | 4.4 | 193 |
| 1600 | 16.4 | 2.6 | 5.2 | 220 |
| 1800 | 16.2 | 2.8 | 5.6 | 248 |
| 2000 | 15.8 | 3.2 | 6.4 | 276 |
| 2200 | 15.6 | 3.4 | 6.8 | 303 |
| 2400 | 15.2 | 3.8 | 7.6 | 331 |
| 2600 | 14.8 | 4.2 | 8.4 | 358 |
| 2800 | 14.5 | 4.5 | 9.0 | 386 |
| 3000 | 14.0 | 5.0 | 10.0 | 414 |
| 3200 | 13.6 | 5.4 | 10.8 | 441 |
| 3400 | 12.8 | 6.2 | 12.4 | 468 |
| 3540 | 11.0 | 8.0 | 16.0 | 489 |

Date 22 Jan 66

Sample No. 15

Temp. of Sample $-10^{\circ}F$.

| Time min. | Dial Reading | Strain per min. | Load |
|---|---|---|---|
| 0 | 700 | | |
| 1 | | | 800 |
| 2 | 634 | | 1000 |
| 3 | | | 1200 |
| 4 | | | 1600 |

Time to Failure 6'

Rate of Strain 0.04"/min.

Thickness 2.584"

$\gamma$ measured 140.5#/cu.ft.

$\gamma$ immersed 142.7#/cu ft

| Load in Kgs. | STRAIN ACROSS SAMPLE | | | Tension psi. |
|---|---|---|---|---|
| | Fiduciary Reading | Divisions Traversed | Strain in. per in. x $10^{-4}$ | |
| 0 | 21.9 | 0 | 0 | 0 |
| 200 | 21.4 | | | |
| 400 | 21.3 | 0.6 | 1.2 | 55 |
| 600 | 21.1 | 0.8 | 1.6 | |
| 800 | 20.8 | 1.1 | 2.2 | 110 |
| 1000 | 20.6 | | | |
| 1200 | 20.3 | 1.6 | 3.2 | 165 |
| 1400 | 20.1 | 1.8 | 3.6 | 192 |
| 1600 | 20.0 | 1.9 | 3.8 | 220 |
| 1800 | 19.8 | | | |
| 2000 | 19.6 | 2.3 | 4.6 | 276 |
| 2200 | 19.4 | 2.5 | 5.0 | 303 |
| 2400 | 19.2 | 2.7 | 5.4 | 330 |
| 2600 | 19.0 | 2.9 | 5.8 | 358 |
| 2800 | 18.7 | 3.2 | 6.4 | 386 |
| 3000 | 18.5 | 3.4 | 6.8 | 413 |
| 3200 | 18.4 | 3.5 | 7.0 | 440 |

Date 13 Feb 66

Sample No. 83

Temp. of Sample - 10$^{o}$F.

| Time min. | Dial Reading | Strain per min | Load |
|---|---|---|---|
| 0 | 750 | | |
| 1 | 729 | 0.021 | 200 |
| 2 | 698 | 0.031 | 400 |
| 3 | 653 | 0.045 | 660 |
| 4 | 604 | 0.049 | 820 |
| 5 | 558 | 0.046 | 1100 |
| 6 | 506 | 0.052 | 1550 |
| | | | 2300 |

Time to Failure 7'45"

Rate of Strain 0.045"/mni

Thickness 2.563 "

$\gamma$ measured 140.5#/cu ft

$\gamma$ immersed 142.5#/cu ft

| Load in Kgs. | STRAIN ACROSS SAMPLE | | | Tension psi. |
|---|---|---|---|---|
| | Fiduciary Reading | Divisions Traversed | Strain in. per in. | |
| 0 | 24.1 | 0 | 0 | 0 |
| 200 | 23.9 | 0.2 | 0.8 | 28 |
| 400 | 23.6 | 0.5 | 1.0 | 55 |
| 600 | 23.1 | 1.0 | 2.0 | 83 |
| 800 | 22.8 | 1.3 | 2.6 | 110 |
| 1000 | 22.4 | 1.7 | 3.4 | 138 |
| 1200 | 22.1 | 2.0 | 4.0 | 165 |
| 1400 | 21.8 | 2.3 | 4.6 | 193 |
| 1600 | 21.5 | 2.6 | 5.2 | 220 |
| 1800 | 21.2 | 2.9 | 5.8 | 248 |
| 2000 | 21.0 | 3.1 | 6.2 | 276 |
| 2200 | 20.8 | 3.3 | 6.6 | 303 |
| 2400 | 20.6 | 3.5 | 7.0 | 331 |
| 2600 | 20.3 | 3.8 | 7.6 | 358 |
| 2800 | 19.9 | 4.2 | 8.4 | 386 |
| 3000 | 19.6 | 4.5 | 9.0 | 399 |
| 3200 | | | | |
| 3400 | 18.8 | 5.3 | 10.6 | 468 |
| 3600 | 18.6/18.1 | 5.5/6.0 | 11.0/12.0 | 482/496 |
| | | | | |
| | | | | |
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| | | | | |
| | | | | |
| | | | | |
| | | | | |

Date 13 Feb 66

Sample No. 84

Temp. of Sample - 10$^{\circ}$F.

| Time min. | Dial Reading | Strain per min | Load |
|---|---|---|---|
| 0 | 473 | | |
| 1 | 448 | 0.025 | 400 |
| 2 | 414 | 0.034 | 520 |
| 3 | 378 | 0.036 | 630 |
| 4 | 340 | 0.038 | 780 |
| 5 | 300 | 0.040 | 970 |
| 6 | 255 | 0.045 | 1280 |
| 7 | | | 1800 |
| 8 | | | 3100 |
| | | | |
| | | | |

Time to Failure 8' 30"

Rate of Strain 0.04"/min.

Thickness 2.537"

$\gamma$ measured 144.7 #/cu ft

$\gamma$ immersed 143.1 #/cu ft

| Load in Kgs. | STRAIN ACROSS SAMPLE | | | Tension psi. |
|---|---|---|---|---|
| | Fiduciary Reading | Divisions Traversed | Strain in. per in. x $10^{-4}$ | |
| 0 | 24.0 | 0 | 0 | 0 |
| 200 | 23.9 | | | |
| 400 | 23.8 | 0.2 | 0.4 | 55 |
| 600 | 23.6 | | | |
| 800 | 23.4 | 0.6 | 1.2 | 110 |
| 1000 | 23.0 | | | |
| 1200 | 22.7 | 1.3 | 2.6 | 165 |
| 1400 | 22.4 | | | |
| 1600 | 22.2 | 1.8 | 3.6 | 220 |
| 1800 | 22.0 | | | |
| 2000 | 21.7 | 2.3 | 4.6 | 276 |
| 2200 | 21.5 | | | |
| 2400 | 21.2 | 2.8 | 5.6 | 330 |
| 2600 | 21.2 | | | |
| 2800 | 21.0/20.8 | 3.2 | 6.4 | 385 |
| 3000 | 20.5 | | | |
| 3200 | 20.1 | 3.9 | 7.8 | 440 |
| 3400 | 19.4 | | | |
| 3600 | 18.8 | 5.2 | 10.4 | 496 |
| 3820 | 14.0 | 10.0 | 20.0 | 525 |

Date 12 Feb 66

Sample No. 109

Temp. of Sample -10°F.

| Time min. | Dial Reading | Strain per min. | Load. |
|---|---|---|---|
| 0 | 462 | | |
| 1 | 452 | 0.010 | 600 |
| 2 | | | |
| 3 | 383 | 0.035 | 940 |
| 4 | 348 | 0.035 | 1060 |
| 5 | 311 | 0.037 | 1250 |
| 6 | 273 | 0.038 | 1700 |
| 7 | | | 2300 |
| 8 | | | 3800 |

Time to failure 8'

Rate of Strain 0.035"/min

Thickness 2.539"

$\gamma$ measured 143.0

$\gamma$ immersed 141.0